American Railway Engineering Association

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Figure 1. Strict environmental controls protect the vegetation on the ARR's right-of-way, as well as the adjacent natural habitat.

AREA'S 1995 FALL TECHNICAL CONFERENCE CONVENES IN ANCHORAGE, ALASKA

The AREA's 1995 Fall Technical Conference was held in Anchorage, Alaska on September 13–15, and proved to be an unqualified success in every respect. Working committee sessions, including the annual Committee Chairmen's meeting, were scheduled for Wednesday. An informative and educational technical session was held on Thursday, September 14, that included eleven excellent presentations on current railway engineering topics. The featured luncheon speaker was former Alaska Governor William Sheffield, who is currently a member of the Alaska Railroad's (ARR) Board of Directors. Attendance at the Conference was exceptionally good, with 196 attendees at the technical session and 112 spouses, for a total registration of 308.

On Friday, September 15, the AREA group boarded a special Alaska Railroad passenger train at Anchorage and traveled through 112 spectacular miles of Alaskan scenery to the historic port of Seward located on the rugged Kenai Peninsula. From Anchorage to Portage, the rail line winds along Turnagain Arm, part of the Cook Inlet known for its 30 foot tides. A 2.4% grade is required to ascend Grandview Hill, as well as five tunnels and curves as great as 14.5 degrees. A multitude of beautiful glaciers and lakes (see cover) are also viewed from the right-of-way. Three unit trains a week traverse this demanding rail line en route to Seward with export coal, in addition to mixed freight trains. Numerous maintenance challenges are also evident, including potential avalanche problems in the winter and unstable permafrost conditions in the spring. In addition, Alaska's strict environmental regulations preclude the use of herbicides for weed control (see Figure 1). ARR's Chief Engineer, Tom Brooks, accompanied the AREA members and guests during the entire trip, commenting on the numerous points of interest and answering many questions. Upon arriving in Seward, the group disembarked for a quick tour of this picturesque fishing harbor and port for a growing number of cruise ships, as well as freighters. Many of the group also opted for an enjoyable luncheon at one of the several waterside restaurants. After the train was turned, all passengers reboarded for the return trip to Anchorage. Cameras were evident everywhere, as everyone scrambled to get the best shots of unlimited spectacular scenery, on both legs of the round trip. Those in attendance will retain fond memories of this outstanding Conference.

Excerpts from Annual Report of Committee 1—Roadway and Ballast

J. R. Zimmerman, Chairman

Status of Subcommittee assignments:

Subcommittee 1—Roadbed

D1-1-93: Foundations for highway-railway grade crossings. A recommended standard practice is being developed and is expected to be completed in 1996.

D1-2-86: Update Manual Sections 1.1–1.4. Improved and current information will be published. Completion is scheduled for 1996.

Subcommittee 2—Ballast

C2-1-86: An ongoing assignment to provide monitoring and recommendations regarding the AAR ballast research program. First draft of HAL ballast tests reviewed and returned. Awaiting second draft.

D2-1-94: Review all FAST and other ballast test data applicable to testing, performance and selection of ballast material. This ongoing project involves the review of a large volume of material to develop potential revisions to the current AREA ballast specifications.

D2-2-89: Study and report on the performance characteristics of hot plant mixed asphalt cemented sub-ballast courses. This ongoing project is proceeding at a slower pace than expected due to insufficient data.

D2-3-89: Study and report on the relationship between in-track performance of ballast materials and laboratory results obtained by the Mill Abrasion test. An information report should be published in 1996.

Subcommittee 4—Culverts

D4-1-94: Update Manual parts 1.4.8, 1.4.9, 1.4.10 and 1.4.15. Expected completion date is 1996.

D4-2-92: Develop Manual recommendations for procedures for culvert inspection. A new section will be completed in 1996.

D4-3-94: Prepare Manual text for proposed part 1.4.18—"Guidelines for Pipe Underdrains." This project is expected to be finished in 1997.

Subcommittee 5—Pipelines

D5-4-92: Prepare specifications for overhead pipeline crossings. Expected completion date is 1996.

D5-1-94: Develop specifications for the use of plastic carrier pipe on railroad right-of-way. This assignment should be completed in 1996.

Subcommittee 6—Fences

D6-1-87: Develop Manual material for security fencing. Work on these advisory recommendations should be complete late in 1996.

D6-1-94: Develop Manual recommendations for sand fences. This ongoing project will apply new technology to the control of sand particles.

Subcommittee 7—Signs

D7-1-91: Review Part 7—Signs for revision. This project has been completed and changes to the Manual will be published in 1996.

Subcommittee 8—Tunnels

D8-1-87: Review and update Manual material. This ongoing assignment is progressing and the upgrading of this section will provide guidance for designers.

Subcommittee 9—Vegetation Control

D9-2-89: Review and update Manual material. This ongoing assignment includes the publication of "as information" articles in the Bulletin, as well as Manual revisions to reflect current practices.

Subcommittee 10—Geosynthetics

D10-1-89: Develop recommended practices for the use of geosynthetics for stabilization of the track roadbed. Completion of this assignment is scheduled for 1996.

D10-1-94: Provide recommendations regarding geosynthetic research to the railroad community. This is an ongoing project.

Excerpts From Annual Report of Committee 2—Track Measuring Systems

R. C. Tannahill, Chairman

Status of Subcommittee assignments:

Subcommittee 1—Track Geometry

D1-2-92: Develop a recommended practice on how to identify and locate priority defects detected by geometry cars. Draft document being developed. Expected completion date is late 1997.

D1-1-93: Track measuring vehicle capabilities. This assignment has been completed and a report was published in the December 1995 Bulletin.

D1-1-94: Develop recommended operating practices for track geometry cars. Expected completion dated is late 1997.

D1-2-94: Study of practices for the storage and use of track geometry car data. This assignment will be completed by the end of 1997.

D1-3-94: Report on relationship between track geometry car measurements and track performance. This task has been broken down into smaller elements, and data is being collected.

Subcommittee 2—Rail Measurement Systems

D2-1-92: Develop a recommended practice for measuring rail parameters. This project is expected to be completed in 1996.

D2-1-93: Develop a recommended practice for the use of automatic rail measurement in the planning of rail maintenance, grinding and renewal. This assignment is expected to be completed at the end of 1997.

Subcommittee 3—Track Structure Assessment

D3-1-92: Develop a recommended practice for gage strength measurement. This assignment was completed and a new section will be published in the Manual in 1996.

D3-2-92: Report on measurement systems related to track lateral stability. This assignment has been delayed due to the concentration of the subcommittee on assignment 1. This project is expected to be completed late in 1997.

Excerpts from Annual Report of Committee 4—Rail

S. A. Atkinson, Jr., Chairman

Status of Subcommittee assignments:

Subcommittee 1—Welding of Rail

D1-1-94: Determine the feasibility of reconstruction of the railhead (metallizing a deposition of a thin layer that reduces wear and/or improves fatigue resistance and/or decreases rolling resistance). This assignment is expected to be completed in 1997.

D1-2-94: Recommend practice for in-track flash butt welding—set minimum criteria and maximum variations. This project should be completed in 1996.

Subcommittee 2—Rolling of the Rail

D2-1-88: Web cracking of rail from roller straightening. This assignment will be completed in 1996.

D2-2-88: Tolerance and method of measuring railhead radius. This project will be completed this year in 1996.

D2-2-93: Produce a standard for tags used on heat treated rail. This project is expected to be completed in 1997.

D2-1-95: Develop a method of permanently identifying industrial grade rails. Expected completion date is 1996.

Subcommittee 3—Rail Statistics

C3-1-76: Rail Statistics. This is an ongoing assignment to provide the industry with a central data collection point.

Subcommittee 4—Thermite and Repair Welding

C4-1-73: Update data on methods and equipment for making welding repairs to rail and turnouts, including thermite welding. This is an ongoing assignment to provide the industry with the latest processes and procedures on rail welding.

Subcommittee 5—Physical Testing and Measurement

C5-1-78: Rail specifications, research and development. This assignment provides a forum for R&D work with rails, rail mills, and other improvements to be discussed.

D5-1-92: Investigate railroad rail specifications to try to standardize on one industry AREA specification. This assignment is expected to be completed in 1998.

D5-1-93: Develop specifications that will cover the microcleanliness of rail steel. This assignment is expected to be completed in 1997.

D5-1-94: Develop standards for track work rails. This assignment is expected to be completed in 1997.

Subcommittee 6—Joint Bars

C6-1-62: Joint bars: design, specification, service test, including insulated joints and compromise joints. This is an ongoing assignment.

D6-1-92: Develop specifications for polyurethane encapsulated joint bars and fiberglass joint bars. This assignment is expected to be completed in 1996.

Subcommittee 7—Grinding, Subrication, AAR and FAST Testing

C7-1-68: Effects of heavy wheel loads on rail. This is an ongoing assignment to keep the industry informed on HAL experience.

D7-1-93: Develop drawings of templates depicting the preferred profiles for optimal rail grinding. This assignment is expected to be completed in 1996.

D7-2-93: Develop recommended practice for measuring lubrication levels on rail heads, also specifying acceptable tribometer levels. This assignment is expected to be completed in 1997.

Subcommittee 8—Non-destructive Inspection

D8-1-93: Develop a drawing of a test rail for ultrasonic purposes. This assignment is expected to be completed this year.

D8-1-94: Investigate different techniques (nondestructive testing-ultrasonics) to determine residual stresses within rails after rolling and in track. This assignment is expected to be completed in 1998.

Excerpts from Annual Report of Committee 5—Track

A. J. Zierow, Chairman

Status of Subcommittee Assignments

1. Assigned Number	B-1-94	C2-1-79	D4-1-92	D4-2-93	C5-1-90	D5-4-81	D5-8-89	D5-2-91
2. Description of Assignments	Pull-aparts	Track Tools	Elast. Fast. Spcl. Trkwk.	Trk. Evalu. Methodology	Update Trkwork. pls.	Explosive Hard Cast	TTC TO. Design	Fstn. for Frog & Cast
3. Assigned Date	12/93	2/79	10/92	12/92	12/90	1/81	5/89	3/91
4. Estimated Comp. percent (%)	0%	Continue	0%	0%	Continue	75%	0%	100%
5. Estimated Completion Date	10/97	Continue	Cancelled 12/3/92	5/97	Continue	5/96	Continue	Completed
6. Problems	None	None	None	None	None	None	None	None
7. Recomm. Assign. (Dropped/Disc.)	No	No	No	No	No	No	No	No
8. Recomm. Assign. (Postpn./Change)	No	No	1/93	1/93	No	No	No	No
9. Subcommittee Chairman	P.I. Cohen	A. Kozak	L. Daniels	L. Daniels	P. Painter	P. Painter	P. Painter	P. Painter
10. Chairman's Comments			Assign. Rolled into D4-2-93 12/92	Turnover to Comm. 30 After re-viewing prototype product at FAST results		Ballot In	AAR/TTC Evaluate	Ballot Pulled for Revisions Passed 5/24/95

Excerpts from Annual Report of Committee 5—Track (cont.)

A. J. Zierow, Chairman

Status of Subcommittee Assignments

1. Assigned Number	**D8-2-84**	**D8-3-85**	**D8-4-89**	**D8-5-94**	**C9-1-92**	**D9-1-92**
2. Description of Assignments	Supr. Elev. & Spril. Lg.	TO. Geomty. Incl. Speed	Vert. Curve Rate/Chg.	Compensated Curve Formulation	Grade Cross. Surfaces	Approaches to Hwy/Rwy Grade X-ings
3. Assigned Date	10/89	10/89	10/89	12/93	2/93	2/93
4. Estimated Comp. percent (%)	10%	15%	100%	100%	Continue	Continue
5. Estimated Completion Date	Continue	10/97	Completed	Completed	Continue	Continue
6. Problems	None	None	None	None	None	None
7. Recomm. Assign. (Dropped/Disc.)	No	No	No	No	No	No
8. Recomm. Assign. (Postpn./Change)	No	No	No	No	No	No
9. Subcommittee Chairman	D. Crouser	D. Crouser	D. Crouser	D. Crouser	K. Autenrieth	K. Autenrieth
10. Chairman's Comments			Ballot out. Passed 5/24/95	Ballot to be Written. Passed 5/29/95	Formerly Comm. 9	Formerly Comm. 9

Excerpts from Annual Report of Committee 6—Buildings and Support Facilities

D. A. J. Perry, Chairman

Status of Subcommittee assignments:

D-3-87: Design criteria for wheel and bearing shops. This assignment has been completed and will be progressed for publication.

D-4-89: Design criteria for material distribution centers. This assignment is almost complete. A draft copy of the report has been distributed to the entire Committee for review and comment.

D-5-89: Design criteria for centralized maintenance of way facilities. This assignment is complete, and the new section was printed in the 1995 Manual update.

D-6-92: Railroad shop noise sources. The committee recommends the dropping of this assignment.

Excerpts from Annual Report of Committee 7—Timber Structures

D. L. McCammon, Chairman

Status of Subcommittee assignments:

C-1-53: Specifications for design of wood bridges and trestles. The subcommittee has prepared a number of graphs and charts depicting E80 stringer loading and pile load distribution. The Committee is now evaluating what information should be included for manual revision. Finalized information should be available for Manual update next year.

C-2-90: Timber technology applications. The format and style of proposed Manual material for Stress Laminated Timber are being revised. Material to be included will be design examples, charts and specifications. Three different species of timber are being specified, with span lengths from eleven to fifteen feet. The proposed material should go to Committee review and letter ballot in 1996.

D-7-94: Effects of heavy axle loads on timber structures. This ongoing project addresses inquiries on ability of timber to withstand HAL's.

D-1-95: Review of current design stress values for timber to insure railroad design values are consistent with other industrial design standards. This project is expected to be completed by 1997.

Excerpts from Annual Report of Committee 8—Concrete Structures and Foundations

G. J. Meyer, Chairman

Status of Subcommittee assignments:

Subcommittee 1—Design of Concrete Structures

D1-1-84: Develop specification for precast and cast in place concrete segmental bridges. Assignment has been approved by full committee ballot, was published as Bulletin information and will be published in 1996 Manual.

D1-1-90: Recommended practice for concrete slab track. Assignment has been approved by subcommittee ballot and entire committee. Will be progressed for publication in 1996.

D1-5-87: Investigate applicable impact factors. A recommendation will be developed this year for a Manual revision.

D1-1-94: Review Part 16—Design of Precast Boxes and Part 13—Design of Cast-in Place Boxes. This assignment will combine and update Parts 13 and 16. Assignment writing has been completed as a metric specification with English units in parenthesis. Has gone to entire committee for ballot and comments.

D1-1-95: Update and metricate Part 2, Reinforced Concrete Design. This project should be completed by the end of 1996.

D1-2-95: Update and metricate Part 17, Prestressed Concrete Design. Completion is scheduled for late 1996.

D1-3-95: Update and metricate Part 19, Rating. This task is scheduled to be completed by the end of 1996.

Subcommittee 2—Design of Retaining Structures

C2-2-90: Design of Proprietary Walls—Review specification to be prepared by Committee 1 for design of such retaining walls.

D2-2-90: Temporary protection for construction. Provides specification for the design of temporary shoring for excavation near a track structure. Progress being made on assignment. Will become a new Part 28 in Chapter 8.

D2-1-94: Revision of Part 5. "Retaining Walls and Abutments." Include specification for scour at bridge piers and abutments in Part 5. Coordinates with subcommittee Assignment D4-4-88. Now ready for Committee vote.

Subcommittee 3—Durability of Concrete and Waterproofing

C3-1-88 &
C3-1-93: Prepare a specification and commentary on cold, liquid applied Elastomeric Membrane. New specification for products used to waterproof bridges. Will be published in Chapter 29 in the Manual in 1996. C3-1-88 is an ongoing assignment to review Chapter 29—Waterproofing.

C3-2-89: Review and rewrite Part 1, as necessary. This assignment is ongoing to review and update Part 1. Letter ballot submitted to entire Committee.

D3-3-90: Prepare Manual material on alkaline aggregate reaction. Approved for ballot by subcommittee.

D3-4-90: Review specifications available on fiber reinforced concrete for possible development of a Manual specification. This assignment will provide specification for use of fiber reinforced concrete. The research has been completed, specification being written.

D3-1-92: Material specification for sulphur concrete. Material prepared and letter ballot issued to the subcommittee.

D3-1-95: Develop new section, Article 23, titled Specialty Concretes, to Part 1 of Chapter 8. Work is progressing.

Subcommittee 4—Repair, Restoration and Strengthening of Concrete Structures

D4-1-91: Commentary for Part 14—Repair and rehabilitation of concrete structures should be completed in 1996.

D4-2-93: Effects of fire on concrete structures. This assignment provides information to assess damage to concrete structures from fire. Should be completed in 1996.

D4-1-94: Update Part 21—inspection. Creates a specification for a new method of constructing a railroad bridge. Project expected to be completed in 1996.

D4-4-88: Develop specification for prevention of scour damage at piers. Complete during 1996.

Subcommittee 5—Design of Concrete Components for Bridges in Seismic Zones

D5-1-89: Develop seismic design criteria or retrofit specifications or guidelines. This assignment was dropped, as it overlaps the work of Committee 9.

Subcommittee 6—Design of Foundations

D6-1-93: Update Part 24—Drilled shaft foundations. Work is progressing.

D6-1-95: Write a history, theoretical basis and/or rationale for some of the specifications in Part 24. This assignment is expected to be completed in 1997.

Excerpts from Annual Report of Committee 9—Seismic Design for Railway Structures

K. L. Wammel, Chairman

Status of Subcommittee assignments:

Subcommittee 1—Post Seismic Event Guidelines and Existing Structures

Subcommittee 2—Basic Concepts, Nomenclature, and Design of Structures

Subcommittee 3—Retrofit Designs

Subcommittee 4—Track, Roadbed, Utilities, Signals, Commuter and Light Rail and Other Miscellaneous Items

Proposed assignments for the above subcommittees have been submitted to the AREA Board for approval.

Excerpts from Annual Report of Committee 11—Engineering Records and Property Accounting

S. D. Jensen, Chairman

Status of Subcommittee assignments:

Subcommittee 1—Accounting

D1-2-90: Study and report on contemporaneous accounting issues applicable to rail operations. This is an ongoing assignment. A proposal was submitted to the ICC, and the Committee is awaiting a response in the form of a Proposed Rule Making.

Subcommittee 2—Planning, Budgeting, Control and Report

D2-2-93: Guidelines are being developed for this new subcommittee, and the Manual is being revised accordingly. This assignment should be completed in 1996.

Subcommittee 3—Taxes

C3-1-88: Study and report on the impact of Federal, state and local taxation issues within the rail industry. This is an ongoing assignment.

Subcommittee 4—Depreciation

C4-1-94: Ongoing project to study and report on the issues involved in depreciating rail and ties—property lives, gross salvage and cost of removal.

Excerpts from Annual Report of Committee 12—Rail Transit

N. Mazzaferro, Chairman

Status of Subcommittee assignments:

Subcommittee 1—Rail Corridor Evaluation

C-1-87: This is an ongoing assignment. There is currently information in the Manual.

Subcommittee 2—Special Track and Roadway Considerations for Rail Transit

C-2-87: Information has been published in both the Bulletin and the Manual. This is an ongoing assignment. A new section on geometry will be published in 1996.

Subcommittee 3—Special Bridge and Structural Considerations for Rail Transit

C-3-87: There is currently material in the Manual, and additional material is being developed at this time.

Subcommittee 4—Rail Transit Electrification

C-4-88: Collaborating as necessary with Committee 33. Material has been developed for a new Part 5 for publication in the Manual in 1996.

Excerpts from Annual Report of Committee 14—Yards and Terminals

T. B. Schmidt, Chairman

Status of Subcommittee assignments:

C-2-88: Collaborations with the Transportation Research Board (TRB) Committee on Intermodal Freight Terminal Design. This is a continuing assignment.

D-8-88: Develop material for Section 4.5 of the Manual on bulk fluids. This assignment was completed in 1995 and is scheduled for publication in 1996.

D-2-89: Develop material for rail-water transfer facilities. This project is expected to be completed in 1996.

D-10-92: Retarders and noise barriers. This assignment is not expected to be completed this year.

D-1-93: Design of waste handling facilities. This assignment is expected to be completed in 1996. Only limited progress has been reported thus far.

D-2-93: Control of railcar speed and routing on conventional ladder tracks (flat yards). A preliminary draft has been submitted to the committee for review. It is expected that this assignment will be completed in 1996.

Excerpts from Annual Report of Committee 15—Steel Structures

J. E. Barrett, Chairman

Status of Subcommittee assignments:

Subcommittee 1—Design Loadings and Stresses

D1-1-87: Obtain data from which the frequency of occurrence of maximum stress in steel railway bridges may be determined under service loading. Further action on this matter awaits the results of the AAR/NSF Bridge Research Program.

Subcommittee 2—Materials

C2-1-92: Review and update fracture control plan. This assignment has been completed and appropriate elements are in the AREA Manual. The fracture control plan will be updated as additional data becomes available.

Subcommittee 3—Fabrication and Erection

C3-1-86: For steel fabrication, develop materials, methods, quality control procedures and qualifications of fabricators. This work is progressing steadily.

Subcommittee 6—Repairs and Maintenance

D6-1-88: Develop methods for repairing damaged steel bridge members. Articles 7.2, 7.4, and 7.5 are currently being rewritten. This assignment is estimated to be completed in 1997.

Subcommittee 7—Special Types of Construction and Miscellaneous

D7-1-92: Develop new Manual chapter on bridge bearings in collaboration with Committees 7 and 8. Part 1, Design, is nearly completed, while Part 2, Construction, will require an additional two years to complete.

D7-1-93: Painting. To prepare recommendations for shop and field coating of new structures, and maintenance and field coating of existing structures. It is estimated that this assignment will be completed late in 1996 or 1997.

Continuous Welded Rail on Bridges: To recommend alternatives for anchoring continuous welded rail on bridges. Work is progressing.

Subcommittee 8—Commentary and Bibliography

C8-1-60 Develop bibliography and technical explanation of various requirements in AREA specifications relating to iron and steel structures. Current issues involve supplemental live loading, design fatigue and metrication. This work is ongoing.

Metric Conversion: Presentations on formula conversions, loadings, and various other portions of Chapter 15 will be dealt with during 1996.

Excerpts from Annual Report of Committee 16—Economics of Railway Engineering and Operation

J. F. R. Gussow, Chairman

Status of Subcommittee assignments:

Subcommittee

D-3-92: Economics of train delay. This subcommittee is currently looking for a new chairman. This assignment will help determine the effects of maintenance on train operations. An interim report will be published in the Bulletin in 1996.

D-4-92: Utilization Considerations of High Performance Locomotives. This assignment is expected to be completed in 1996.

D-5-92: Renewal with concrete ties, complete or part. This assignment will assist in establishing the economics of installing concrete ties. Searching for new chairman.

D-8-92: Track time usage. This assignment is 30% complete and is in need of a new subcommittee chairman.

D-1-94: Remote control locomotive technology. Work temporarily suspended pending FRA rule-making on the subject.

Excerpts from Annual Report of Committee 17—High Speed Rail

H. H. Moehren, Chairman

Status of Subcommittee assignments:

Subcommittee

D-1-89: Corridor evaluation. This assignment was completed in 1994 and was published in the 1995 AREA Manual.

D-1-95: Development of information and recommended practices regarding shared rail, passenger and freight with speeds more than 110 mph, but less than 150 mph.

C-1-89: Track structures and track train interactions. An outline of this material was published in the AREA bulletin in 1994. Completion is scheduled for late 1997.

C-2-89: Vehicles, control, and propulsion system considerations for high speed rail, collaborating on electrification with Committee 33—Electrical Energy Utilization, and the AAR Communication and Signals Division. This is an ongoing assignment, an outline of which was published in the AREA bulletin in 1994. A new Part 3 will be published in the 1996 Manual.

Excerpts from Annual Report of Committee 18—Light Density and Short Line Railways

R. E. Larsen, Chairman

Status of Subcommittee assignments:

Subcommittee 1

C1-1-90 Recommend practices concerning the use of secondhand track material, including inventory of rail and OTM 100 and less. This is an ongoing assignment. Specifications for secondhand (relay) rail are currently being established. This assignment is expected to be completed in 1996.

C1-2-90: Recommend practices for obtaining, maintaining, organizing and operating maintenance of way work equipment of shortlines. This is an ongoing assignment.

Subcommittee 2

D2-1-90: Investigation of the special considerations for obtaining contracted engineering, repair, and maintenance services for shortlines.

D2-2-90: Provide inspection criteria for track and bridges to shortlines. Expected completion in 1996.

D2-3-92: Compile a list of the vital documents which describe the engineering assets of a railroad collaborating with Committee 11.

D2-4-92: Develop recommended practices for budgeting and planning the engineering work of short line and light density lines, collaborating with Committee 16.

Table of Rail Sections. A new and expanded table of rail sections was prepared for publication in the Portfolio of Trackwork Plans in 1996.

Excerpts from Annual Report of Committee 24—Engineering Education

C. B. Congdon, Jr., Chairman

Status of Subcommittee assignments:

Subcommittee 1—Continuing Education

C1-1-71: Continuing Education. This is an ongoing assignment to improve the overall knowledge of AREA members.

Subcommittee 2—Student Relations

C2-1-82: Recruiting of engineering student graduates. This is an ongoing assignment.

C2-2-88: Student Interest Award Program. This is an ongoing assignment. The winner of the 1994 SIAP, Eric Stump, had his paper published in the May 1995 Bulletin.

C2-3-71: Student relations. This is an ongoing assignment to promote the Student Affiliate Program.

Subcommittee 3—Faculty Relations

C3-1-83 Faculty relations. New projects are being developed.

C3-2-82: Speakers for student groups. Speakers at AREA Technical Conferences are being contacted with respect to furnishing copies of their presentations to professors. This is an ongoing assignment.

Excerpts from Annual Report of Committee 27—Maintenance-of-Way Work Equipment

C. W. Jones, Chairman

Status of Subcommittee assignments:

Subcommittee

C-1-74: Reliability engineering as applicable to work equipment. This is an ongoing assignment. Efforts are currently focusing on standardizing definitions and measurement of reliability and equipment down time.

C-2-83: Preventive maintenance for maintenance of way equipment. Subcommittee has proposed a number of reports outlining different maintenance procedures.

D-2-86: Computer applications within work equipment organization. Progress on the assignment continues with papers presented and demonstrations given on various rail and vendor computer systems.

C-3-92: Maintenance of work equipment safety and ergonomics. A workshop was presented in October 1994 addressing equipment ergonomic issues. Related topics are discussed at each Committee meeting.

D-1-77: Training programs for machine operators and maintenance personnel. This committee is ongoing and is currently pursuing a library of available videotapes and other training resources for all sources of information.

Excerpts from Annual Report of Committee 28—Clearances

P. M. Williams, Chairman

Status of Subcommittee assignments:

Subcommittee

C1-1-62: Compilation of the railroad clearance requirements of various states. This is an ongoing assignment. All states have been updated and a new version is being finalized. Completion is scheduled for 1996.

C-2-85: Compilation of a comprehensive glossary and bibliography pertaining to the technical literature on railroad high and wide clearances. This is an ongoing assignment. The subcommittee is planning to print out a loose leaf format of terms to be given to interested parties.

C-3-85: Review of Railway Line Clearances to develop improved user accessibility. A proposed new format has been submitted to the publisher, however, the future of this publication has not been established. This project should be completed in 1996.

C-4-91: Research, report and provide equipment clearance diagrams (plates) as required. Further study of equipment vs. plates is being undertaken.

C-5-92: New designs in intermodal containers and their effect on clearances. The subcommittee is presently researching major car manufactures to determine what kind of future equipment is being considered.

D-3-85: Conversion of "Heavy Capacity and Special Type Flat Car" section of the Official Railway Equipment Register to UMLER compatible format. A review of all heavy duty car types has been undertaken and a listing of odd car footprints has been compiled. A final report is anticipated at an early date.

D-4-85: Research and develop book covering heavy duty car diagrams and ratings. This information will be published in 1996.

D-6-89: Recommendations for a uniform electronic clearance message. This subcommittee is currently working with the TG5 committee to develop a standard clearance transaction set for the rail industry.

D-7-92: Clearances for highway structures over railroads. The diagrams and write-up are being finalized. They should be ready in 1996.

Excerpts from Annual Report of Committee 30—Ties

J. F. Scott, Chairman

Status of Subcommittee assignments:

Subcommittee 1—Cross Switch, Bridge and Crossing Ties

C1-2-63: Keep up-to-date specifications for all types of ties. This is an ongoing assignment.

C1-1-62: Investigate possible revision of crosstie design and/or spacing. This assignment is monitoring performance with respect to 286 and 315 kip cars.

C1-4-94: Develop complete transit tie requirements, collaborating as necessary with Committee 12. Work is progressing and completion of the project is scheduled in 1998.

Subcommittee 2—Wood Preservatives and Preservative Treatment of Forest Products

C2-1-63: Keep up-to-date specifications for preservatives and update research for new preservatives. This is an ongoing assignment.

C2-2-63: Keep up-to-date specifications for seasonings, conditioning and treatment. This is an ongoing assignment.

C2-3-63: Advisability of preparing Specifications to cover care and handling of forest products before and after treatment. This is an ongoing assignment.

Subcommittee 3—Collaborate with AAR Research and Test Department and Other Organizations in Research and Other Matters of Mutual Interest

C3-1-76: Substitute for sawn-wood ties. This is an ongoing assignment.

C3-4-82: Monitor progress of each subcommittee and assist with an appropriate exchange of technology between AAR and AREA. This is an ongoing assignment.

C3-2-94: Monitor progress of AAR Rail Seat Abrasion Committee. Completion is expected in 1997.

C3-3-94: Monitor progress of AAR tie research.

Subcommittee 4—Flexural Strength Requirements

C4-1-87: Monitor developments in prestressed and reinforced concrete technology which may affect concrete tie requirements. This is an ongoing assignment.

D4-2-89: Investigate impact resistance and design requirements for ties. This project is expected to be completed in 1997.

D4-3-87: Evaluate acceptance criteria for repeated load tests. This project is in progress with no completion date specified.

Subcommittee 5—Fastenings

D5-1-83: Revise current test requirements.

D5-2-83: Investigate the effects of axle loads and tie spacing on fastening requirements.

D5-3-86: Review and recommend revisions of the load magnitude specified for the fastening repeated load tests. There has been good progress on this assignment to date. No completion date has yet been specified.

Subcommittee 6—Durability

D6-1-89 Resistance of concrete ties to alkali aggregate reaction.

D6-2-89: Resistance of concrete ties to freezing and thawing. The best concrete mix designs will be identified.

D6-3-89: Resistance of concrete ties to rail seat abrasion, including during construction. Extensive on site testing has resulted in recommended procedures for combating this problem. This is an ongoing assignment.

Subcommittee 7—Maintenance Requirements

C7-1-87: Maintenance requirements of concrete ties, including pads and insulation. Will collaborate with Committee 5.

Subcommittee 8—Tie Disposal

D8-1-89: Tie disposal alternatives.

Excerpts from Annual Report of Committee 32 -Systems Engineering

M. C. Walbrun, Chairman

Status of Subcommittee assignments:

Subcommittee 2—Engineering Management Systems. Review, research and disseminate information pertinent to design and implementation, including specific applications or techniques within the scope of railroad engineering.

C2-1-86: Research new applications for the use of information technology that will assist in the management of engineering resources. This is an ongoing assignment. Subcommittee research results are usually disseminated via Committee-sponsored symposiums, presentations at Committee meetings, proceedings articles and March conference presentations. Research results also are forming a substantial base for the development of Manual material.

D2-1-92: Initiate the development of Manual material for Engineering Management Systems. Progress has been made on the development of a Table of Contents for Manual Chapter 32, which was published in 1995 and will serve as an outline for future tasks. A reformatted table of contents and foreword will be published in 1996, together with a new section on mapping in Part 11, Computer Aided Drafting.

Subcommittee 3—Systems Engineering Education. Collect and disseminate information to the Association membership by means of special features, demonstrations and printed material.

C3-1-78: Application of engineering information systems as related to design and analysis problems and maintenance practices.

Subcommittee 4—Engineering Graphics Systems and Interchange Standards. Collect information from North American Railroads on graphic systems uses and examine existing drawing transfer standards.

D4-1-86: Develop a Manual section with recommended protocol for CADD drawings. Work is continuing on this assignment. The subcommittee has disseminated surveys to determine the types of CADD equipment and systems used on the various roads. Several CAD drawings, including some of the initial drawings prepared by the consultant for the *Portfolio of Trackwork Plans* have been transmitted between several railroads and tested to determine the effectiveness of transfer techniques and protocols. Due to the rapid advance in technology this investigation and the survey of the various railroads must be updated every few years. It is difficult to make recommendations without favoring a specific popular equipment vendor. The subcommittee is, however, making progress and completion of this assignment is scheduled for 1997.

The Committee has recommended that the title of Chapter 32 be changed to Engineering Management Systems.

Excerpts from Annual Report of Committee 33—Electrical Energy Utilization

P. K. Stangas, Chairman

Status of Subcommittee assignments:

Subcommittee 1—Electrification Economics

C1-1 73: A review of current projects and their associated economic aspects will be initiated and a report prepared for possible inclusion in the Manual.

Subcommittee 4—Railroad Electrification Systems

D4-1-87: This subcommittee is actively engaged in a review of the present manual section and some changes may result. A revised draft is likely to result over the next few months. There appears to be a need to coordinate the technical content and details with Committees 12 and 17. This review will also result in identification of specific topics of coordination. This subcommittee has recently incorporated some changes in the 1995 edition of the Manual.

Subcommittee 6—Power Supply and Distribution

C6-1-87: This subcommittee has recently completed a revision to this section of the Manual (Part 6), which will be published in 1996.

Subcommittee 8—Equipment Generated Electrical Noise

D8-1-86: This subcommittee has been inactive. However, the issue remains one of interest and perhaps one of increasing concern in light of new communications-based technologies and new high speed equipment under consideration.

Excerpts from Annual Report of Committee 34—Scales

R. K. Feezor, Jr., Chairman

Status of Subcommittee assignments:

Subcommittee

C-1-85: Preparation of subjects for publication. This is an ongoing assignment.

C-2-82: Innovations in scales used in connection with operations of railroads. This assignment evaluates new technologies to determine their relevance and suitability pertaining to application and cost effectiveness in a railroad environment.

D-2-87: Investigate stenciling of cars using coupled-in-motion weights. The final report for this assignment was forwarded to the AAR TSS Committee for comments. Awaiting outcome of Federal and state regulatory action.

D-3-90: Track scales testing guidelines, test and inspection forms. Recommends uniform reporting criteria. The static report is complete. Work continues on the motion report.

D-4-91: Railroad master scale program. This assignment will provide a condition evaluation report for railroad master scales. It is currently being reevaluated.

Proposed 1996 AREA Manual and Portfolio Revisions

The following proposed Revisions of the AREA *Manual for Railway Engineering and Portfolio of Trackwork Plans* have been recommended to the association by the technical committee responsible for each after a letter ballot is approved by: (1) a two-thirds majority of the eligible members voting, and (2) by at least fifty percent of the total eligible voting members on the committee. They are being published here for comment by the general AREA membership and any other interested parties. Comments should be sent to AREA headquarters by March 1, 1996. These comments will be considered by the AREA Board of Direction in deciding whether to give final approval for inclusion of the proposed changes in the Manual and Portfolio Revisions, which if approved, go into effect August 1, 1996.

Proposed 1996 Manual Revisions to Chapter 1—Roadway and Ballast

Part 2—Ballast

Page 1-2-6.6, Article 2.11. Replace current article with the following:

SUB-BALLAST SPECIFICATIONS

2.11 SUB-BALLAST SPECIFICATIONS

This part of the specifications cover design, materials and construction of the sub-ballast section laying between the track ballast and the subgrade as defined in Article 2.0.2.4, and composed of a section of smaller dense or well graded granular material. Sub-ballast material is primarily used for the construction of new tracks.

General

For over 50 years railroad construction and maintenance practices have utilized a roadway structure for heavy traffic composed of a ballast section approximately 24 inches in depth that included both track ballast and sub-ballast. Experience has indicated a substantial portion of this ballast depth may be successfully composed of a compacted sub-ballast material also serving as a buffer or filter to prevent subgrade material from penetrating the sub-ballast section while at the same time permitting water from whatever source to escape from the area of the subgrade surface. Discussion of the functions of the sub-ballast is provided in the commentary. The engineer must follow established engineering principles for the design, selection of materials and construction of the sub-ballast section of the track substructure.

2.11.1 Design

The railroad substructure must be designed so that the subgrade, sub-ballast and track ballast provide uniform support and distribution of superstructure loadings. The subgrade strength will dictate the combined depth of ballast and sub-ballast materials.

The following conditions should be considered in the design of the sub-ballast section:

1. Engineering properties of subgrade soil.
2. Support capability of subgrade.
3. Unit load applied to the ballast at the base of tie.
4. Total thickness (track ballast + sub-ballast).

5. Sub-ballast properties.
6. Gradation of sub-ballast.
7. Installation and compaction.

2.11.1.1 Subgrade Soils

The minimum data needed to evaluate the subgrade soils should be classification (which requires Atterberg limits and gradation as appropriate) and strength (lowest expected). The depths and thicknesses of the lower strength layers to a depth of at least two feet should be examined. The following current ASTM test designations may be used in developing the necessary data where appropriate for design:

Plastic Limit and Plasticity Index	D4318
Grain Size Analysis	D421 (Sample Preparation)
	D422 (Test Procedure)
Compaction Test	D698
	D1557
Unconfined Compression Test	D2166

Where cohesive soils exist in the subgrade, resulting of an unconfined compression test of the compacted cohesive material (saturated) will give a cohesion or shear strength for use in design. It may not be necessary to develop shear values from tests for some non-cohesive soils but where necessary standard tests may be performed. In absence of testing, caution is advised in applying the AREA allowable bearing pressure of 20 psi for design from Chapter 22.

The level of stress in the subgrade should not exceed an allowable bearing pressure that includes a safety factor. A minimum factor safety of a least 2 and as much as 5 or more should be provided to prevent bearing capacity failure or undue creep under the loaded area. When subgrade support is marginal and/or where the liquid limit of the subgrade soil exceeds a value of 30 or the plasticity index exceeds 12, special attention should be given to that soil. A change of subgrade soil or stabilization of the subgrade material may be considered to obtain a more reliable support for the sub-ballast.

2.11.1.2 Loads Supported by Track Ballast

Many variables affect the stress placed by the wheel load on the tie and the load is distributed over many ties.

Example calculation follows:

Problem: Develop ballast depth below base of tie for a proposed track supporting a 36 inch diameter wheel (36000 pound wheel load) at speed of 55 mph, 136 lb. rail and 7" x 9" x 8'-6" oak ties @ 21" spacing. Assume a saturated subgrade support value of 18 psi includes a safety factor of 2.

The AREA impact factor for track: 33V/100D where V = velocity in mile per hour and D = diameter of the wheel.

Impact Factor = (33 x 55) / (100 x 36) = 1815/3600 = 0.50.

Distribution Factor: For 21 inch tie spacing 47% of axle load is assumed applicable to each tie either side of the applied load. (Arrived at by using Talbot distribution utilizing a track modulus of 3500 lb. per inch per inch).

AREA formula for average ballast pressure (psi) at tie face:

ABP = [2P (1 + IF/100) (DF/100)]/A

where

P = 36000 (Wheel loading in lbs.).
IF = 50 (Impact factor in percent).
DF = 47 (Distribution factor in percent).

A = 918 Area of face of 7" x 9" x 8' – 6" ties in sq. in.
ABP = Average ballast pressure at base of tie.
ABP = [2 x 3600 x (1 + 50/100) (47/100)]/918 = 55 psi.

2.11.1.3 Depth of Ballast Plus Sub-Ballast

The distribution of loads to depth is approximately the same regardless of the granular material. Therefore the combined depth of sub-ballast and ballast is calculated as a single unit to develop the pressure on the subgrade. Talbot developed an empirical formula for vertical pressure exerted by the ballast under the tie at its intercept with the rail at a depth below the bottom surface of the tie.

$$p_c = 16.8\ p_a/h^{1.25}$$

Where

p_a = bearing pressure on subgrade including safety factor.
p_c = uniformly distributed pressure over tie face.
h = depth below face in inches.

If the tie pressure p_a in pounds per square inch and the bearing capacity of the subgrade p_c are known, the minimum depth of ballast in inches required to produce a stable structure is:

$$h = (16.8\ p_a/p_c)^{4/5}$$

Assuming an allowable subgrade pressure of 18 psi (a safety factor of 2) and using the unit tie face pressure developed above of 55 psi, solve for ballast depth:

$$h = (16.8 \times 55/18)^{4/5} = (924.0/18.0)^{4/5} = 23.4 \text{ inches}$$

The capacity of the subgrade including the safety factor must always be equal to or greater than the load placed upon it.

2.11.1.4 Location of Ballast–Sub-Ballast Interface

The sub-ballast layer depends upon its state of compaction to be most effective. The present specified depth of 12 inches of track ballast below the tie precludes the maintenance tamping from penetrating and damaging the sub-ballast layer. The force calculated by the above formula for a point 12 inches beneath the tie is 41.4 psi, a force that will reduce tendency of larger ballast particles to penetrate the sub-ballast. The remaining depth of required ballast is furnished by the sub-ballast.

2.11.1.5 Sub-Ballast Materials

Material most commonly available for use as sub-ballast are those aggregates ordinarily specified and used in construction for highway bases and subbases. These include crushed stone, natural or crushed gravels, natural or manufactured sands, crushed slag or a homogeneous mixture of these materials. Other natural on site materials conforming to proper engineering standards and specifications as may be defined by individual railway companies may be used.

The sub-ballast shall be a granular material so graded as to prevent penetration into the subgrade and penetration of track ballast particles into the sub-ballast zone. Applying the filter principle used in drainage to the grading of the subgrade material will determine the grain size distribution of the sub-ballast. Most state highway specifications include standard gradations for dense graded aggregate (DGA) and aggregate base course (ABC). These gradations may meet the requirements for use as sub-ballast. Other standard gradations may also meet these requirements.

Prepare the gradation curve for the sub-ballast by plotting the grain size distribution for the subgrade on a semi-logarithmic paper, using the logarithmic scale for the grain sizes and the natural scale for percent passing. Determine the grain-sizes at 15%, and 50% points on the chart. Use these values with relevant ratios from Table 2.11.1 to compute the limiting grain sizes at the 15% and 50% passing lines on the chart. The maximum grain size of the sub-ballast must not exceed the maximum grain size of the track ballast. No more than 5% of the sub-ballast should pass the No. 200 sieve. Construct

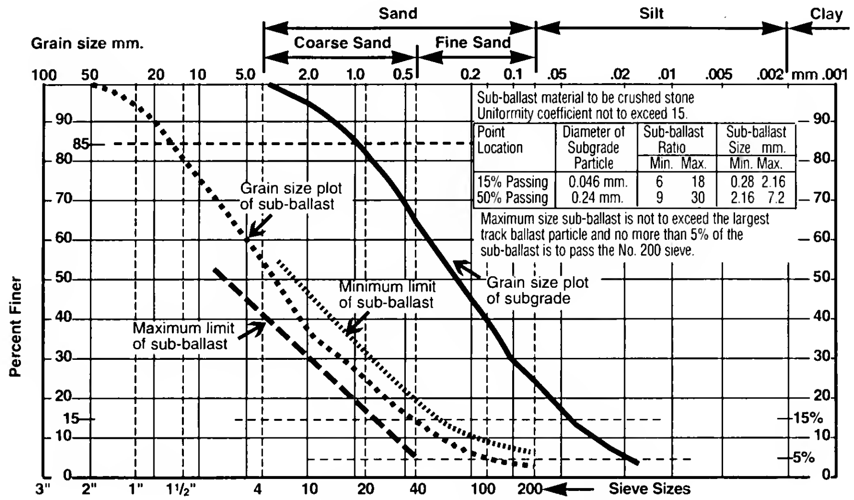

Figure 2.11.1. Example using Table 2.11.2.

Table 2.11.1. Requirements for Filter Material (after USBR1963)

Character of Filter Materials	Ratio R_{50}	Ratio R_{15}
Uniform grain-size distribution (U = 3 to 4)	5 to 10	—
Well graded to poorly graded (non uniform); subrounded grains	12 to 58	12 to 40
Well graded to poorly graded (non-uniform); angular particles	9 to 30	6 to 18

$R_{50} = \dfrac{D_{50} \text{ of filter material}}{D_{50} \text{ of material to be protected}}$ $R_{15} = \dfrac{D_{15} \text{ of filter material}}{D_{15} \text{ of material to be protected}}$

Note: Grain-size curves (semilogarithmic plot) of sub-ballast and subgrade should be approximately parallel in the finer range of sizes.

The above Table 2.11.1 was prepared especially for earth dam design and since the use here is for a different purpose the values given may be slightly exceeded.

In event that soil in subgrade may be subject to piping, position the maximum percentage value of D for the sub-ballast to be less than 5 x D_{85} of the subgrade soil The sub-ballast in this case should be well graded.

lines connecting the minimum and maximum points to set limits for the sub-ballast material. See example Figure 2.11.1.

2.11.2 Testing

Some of the most frequently used tests for sub-ballast material are given in Table 2.11.2 which state properties, test methods, and comments on limiting values.

Table 2.11.2. Sub-Ballast Properties and Test Methods

Property	Test Method	Comments
Particle Size Analysis	ASTM D 422	See Section 2.11.1.5
Moisture Density Relation	ASTM D 1557	Maximum Dry Density and Optimum Moisture Content
Liquid & Plastic Limits Minus No. 40 Sieve	ASTM D 423 D 424	See Design Section
Degradation—Los Angeles Abrasion	ASTM C 131	Variable*
Sodium Sulphate Soundness	ASTM C 88	Variable*
Percent Material Passing No. 200 Sieve	ASTM C 117	Variable*
Permeability	ASTM D 2434	Variable*

*The numerical value of these tests will depend upon the physical and chemical characteristics of both the ballast and subgrade as well as the material used for sub-ballast and values as may be defined by the individual railway companies.

2.11.3 Construction of Sub-Ballast Section

The subgrade shall have been graded, shaped and compacted as required by the plans and specifications. The top of the subgrade require special attention to obtain uniform density. A uniformly smooth surface compacted to specifications is required, containing no ruts, pot holes, loose soil or any imperfection retaining water on the surface. The surface shall be inspected by the engineer and if surface fails to conform to specifications the engineer may require blading, rolling and compacting to provide a satisfactory surface.

The sub-ballast material shall be transported and delivered to the site in a manner that will prevent segregation or loss of material. The material shall be placed in layers of 3" to 6" (or as directed by the engineer) and compacted to depth and density as required by the plans and specifications. The sub-ballast shall be shaped as required by the plans and specifications and the finished surface shall be free from surface defects and imperfections that will retain water or restrict free flow of water.

Vehicular traffic is to be kept to a minimum across the newly prepared sub-ballast surface. The contractor shall be responsible for maintaining a firm, true and smooth surface compacted to the required density until track ballast is placed on the sub-ballast.

2.11.4 Production and Handling

Production and handling shall conform to Article 2.5 of this chapter for track ballast.

2.11.5 Inspection

Inspection of material at production site shall conform to Article 2.7 of this chapter.

2.11.6 Measurement and Payment

The pay item for furnishing, placing, shaping, compacted and maintaining the sub-ballast until acceptance by the railway shall be "sub-ballast" and the pay unit shall be by the ton.

Measure and payment for water used to moisten subgrade prior to placing the sub-ballast, in mixing sub-ballast material to maintain proper moisture during compaction and maintenance of the surface during construction shall not be measured for separate payment but shall be considered incidental to sub-ballast payment.

Part 7—Roadway Signs

Page 1-7-1. Replace current Part 7 with the following:

ROADWAY SIGNS[1]
1995

7.1 CLASSIFICATION

7.1.1 Location

7.1.1.1 Mile Post

Provide a ready method of location of any physical object on right of way.

7.1.1.2 Political Subdivision Signs

Show where railways cross national, state, county and municipal boundary lines.

7.1.1.3 Standard Right of Way Sign and Monument Marker

Define railways rights of way limits.

7.1.1.4 No Trespassing Signs

Define locations where trespassing is especially unsafe and/or undesirable.

7.1.2 Maintenance of Way

7.1.2.1 Maintenance Limit Signs

Define limits of track ownership and maintenance between railway and industry and between railways.

7.1.2.2 Roadway Structures Signs

Define location of bridges, trestles, tunnels and culverts.

7.1.2.3 Snowplow Signs

Define locations of obstructions to snow removal equipment. Flanger signs warn operators to lift flanges. Wing markers warn operators to close snowplow wings. If both indications are required at the same location they should be on one (1) sign.

7.1.2.4 Alignment Signs or Markers

Define the exact locations and limits of easement spirals and curves. Superelevation should be shown on the sign or marker at the spiral curve point.

7.1.2.5 Elevation Markers

Define top of rail elevations at special locations. Also define top of rail elevations above which tracks under grade separations cannot be raised.

7.1.3 Transportation

7.1.3.1 Speed Control Signs—Temporary and Permanent

Define limits of slow orders and locations to stop trains.

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[1]References, Vol. 40, 1939, pp. 536, 729; Vol. 52, 1951, pp. 481, 809; Vol. 53, 1952, pp. 698, 1106, Vol. 54, 1953, pp. 1092, 1385; Vol. 63, 1962, pp. 579, 749; Vol. 80, 1978, p. 79.

7.1.3.2 Whistle Posts

Define advance location of highway grade crossings, stations, railway grade crossings and other special locations where locomotive whistles are required to be sounded.

7.1.3.3 Location Signs

Define advance location of railway grade crossings, drawbridges, tunnels, junctions, frequent rock and snow slides, and stations. Also used to define yard limits, switching limits, signal territory limits, station and derail location track capacities, jointly owned track and start or end of ownership.

7.1.4 Safety

7.1.4.1 Restricted Clearance Signs

Identify locations of restricted horizontal and vertical clearances or both, at clear points of turnouts, buildings, platform or other structures.

7.1.4.2 Fire Hazard Signs

Warn everyone concerned of flammable materials storage and subgrade carriers. These signs must meet latest Code Federal Regulation Title 29, Part 1910.

7.1.4.3 Electric Hazard Signs

Warn everyone concerned of the presence of overhead and subgrade high voltage carriers. These signs must meet latest Code Federal Regulation Title 29, Part 1910.

7.1.4.4 Highway Grade Crossing Warning Signs

Warn all vehicular traffic of the presence of a railway grade crossing. All highway grade crossing warning signs will conform to the current U.S. Department of Transportation manual on uniform traffic control devices. Also all applicable current State Department of Transportation Specifications must be followed.

7.1.4.5 Barricade Signs

Used to warn all vehicular traffic of a railway grade crossing under construction or repairs.

7.1.4.6 Highway and Barricade Signs

Must meet current U.S. Department of Transportation manual uniform traffic control devices and applicable State Department of Transportation Specifications.

7.1.4.7 Power Operated Switch Signs

Used to warn pedestrian traffic of the presence of a power operated switch. This sign is especially desirable when non-railroad pedestrian traffic will pass close to a power operated switch.

7.2 DESIGN

7.2.1 Shapes

Definite sign shapes allow rapid identification.

7.2.2 Dimensions

Dimensions of various groups of signs may differ within limits determined by the legend.

7.2.3 Background

The sign background in sharpest contrast with the lettering is best. Black letters on "white" or "yellow" background show well, however, the background may be varied to conform with local conditions. Backgrounds on speed control signs should conform in color to indications on the particular railway.

7.2.4 Legends

Legends on signs should be short, consisting of characters that are large, plain and widely spaced for legibility at the maximum required distance. Wording on signs should be minimized. Proper spacing of characters should be determined by field tests. Bold stroked letters are preferable. The same characteristics are valid for day and night operations.

7.2.5 Placement

Terrain background behind signs merits consideration. To be effective signs must be prominently displayed. Signs should be placed to clear all maintenance of way equipment.

7.3 MATERIALS

7.3.1 Wood

Only preservative treated wood is recommended for posts. The use of untreated dense tropical hardwoods may be considered. Also, all State and Federal Environmental Regulations must be observed when and where preservative treated wood is used as sign post material.

Wood posts are usually tamped solid into ground.

7.3.2 Concrete

Concrete is recommended as a metal sign post base. Waterproofed reinforced concrete is a satisfactory post material.

7.3.3 Reflectorized Materials

Black letters on "white" or "yellow" reflectorized background on either aluminum plate or extruded aluminum make an excellent sign. Fiberglass signs with moulded letters also make a serviceable sign.

7.3.4 Metals

Galvanized steel channels and rods with sharpened ends make serviceable posts. The metal sign posts are especially useful for temporary signs. A break away post may be considered where steel is used.

Part 10—Geosynthetics

Page 1-10-16. Add new Article 10.4, as follows:

10.4 GEOCOMPOSITE DRAINAGE SYSTEM SPECIFICATIONS FOR RAILROAD APPLICATIONS

10.4.1 Introduction

10.4.1.1 Significance and Use

The use of geocomposite drainage systems for track roadbed stabilization applications is dependent on environmental and subgrade conditions. Geocomposite drainage systems shall be used when necessary to provide for the removal of groundwater and relief of hydrostatic pressure within the trackbed, subgrade soil and adjacent areas.

Geocomposite drainage systems have numerous civil engineering applications apart from the application described herein. Examples of these applications include turf/slope interceptor drains, retaining wall drainage, foundation drainage, roof drains and other subsurface water collection and removal applications.

10.4.1.2 Applications

This work shall consist of furnishing and installing a geocomposite drainage system as a track roadbed subsurface drain. The geocomposite drainage system shall be designed to allow for efficient collection and disposal of water while retaining the in situ soil, without clogging and/or blinding the geotextile and the core shall be designed to resist deformation, fabric intrusion and reduced flow capacity due to heavy static and dynamic railroad loads.

The use of geocomposite wick drain systems require that a complete soil investigation be made by a geotechnical engineer to determine the required flow capacity, filtration requirements and drain spacings. Geocomposite wick drain systems are not covered under this specification.

10.4.2 Material Requirements

The prefabricated geocomposite drainage system shall consist of a flexible three dimensional synthetic drainage core. The drainage core shall be tightly encapsulated by a nonwoven geotextile. The core shall consist of a sufficient number of support members for composite support interaction between the drainage core and the geotextile overwrap to prevent geotextile intrusion and in-plane flow reduction.

10.4.2.1 Testing and Design

The long-term performance of the geocomposite drain core element shall be compatible with site-specific loadings acting upon the buried drain. The manufacturer shall provide independent test results from a qualified laboratory showing the incremental compressive creep behavior over time of the drainage core when subjected to continuously-applied pressure at equivalent to or greater than the in-service loadings determined by the design engineer.

The long-term compressive creep behavior tests may be conducted for a minimum period of 10,000 hours at 20°C. All product samples used for long-term creep deformation testing shall be indexed to their short-term compressive strength properties from quality control records. All product samples used in compressive creep testing shall be representative of the product to be supplied.

The required in-plane flow capacity of the geocomposite drainage system shall be based on the site specific drainage requirements. Independent in-plane flow test results on representative product samples shall be provided to the designer by the geocomposite drainage system manufacturer. These test data shall include in-plane flow results in a soil environment at load levels, gradients, time intervals, and outlet spacing requirements consistent with the anticipated site specific loadings and conditions to provide the designer with insight into the long-term in-plane flow capacity of a particular product being considered for use. ASTM D4716, Test Method For In-Plane Flow Capacity may be used as a comparison of in-plane flow capacity.

10.4.3 Geotextile Overwrap

The geotextile overwrap shall be a nonwoven, needlepunched geotextile and conform to the properties outlined in Table 10.2.1 Class A or Class B to meet the installation survivability standards of the application as determined by the engineer.

10.4.4 Packing and Identification Requirements

The geocomposite drainage system shall be provided in rolls wrapped with a protective covering. A tag or other method of identification shall be attached to each wrapped package indicating the following:

1. Manufacturer or Product Name
2. Date of Manufacture of Product
3. Roll Identification Number
4. Width of Product Rolls
5. Length of Product Rolls

10.4.5 Compliance, Inspection and Sampling Requirements

A competent laboratory must be maintained by the producer of the geocomposite drainage system, at the point of manufacture, to insure that quality control is in accordance with ASTM testing procedures. That laboratory shall maintain records of its quality control results and provide, upon request of the specifying agent prior to shipment or at any other reasonable time thereafter, a manufacturer's certificate or actual test roll data. The certification shall be based on minimum average roll values, and shall include:

(a) Name of Manufacturer

(b) Chemical Composition

(c) Product Description

(d) Statement of Compliance to Specification Requirements

(e) Laboratory Index Test Results From the Lots Corresponding To the Rolls Shipped

(f) Long-Term Compressive Creep Behavior and In-Plane Flow Test Results

(g) Signature of Legally Authorized Official Attesting to the Information Required

(h) Geotextile Overwrap Properties

It is recommended that product samples be taken at random or at an interval specified by the Engineer in the field, and be tested at an independent competent laboratory to indicate compliance with these specifications.

10.4.6 Construction Details and Methods

The geocomposite drainage system shall be installed in accordance with the plans and specifications or as directed by the engineer.

A geocomposite drain system shall be placed in a trench as shown in Figures 10.4.6. Trench depths, widths and slopes shall be as shown on the plans or as determined by the engineer. The excavated trench material may be used for backfilling the geocomposite drain system. If required, the drain shall be temporarily supported to assure vertical alignment while the backfill is being placed. The backfill operation shall take place in a minimum of two lifts with a maximum lift thickness of

8". The final lift shall bring the backfill to an elevation approximately two Inch (2") over the top of the drain. Care shall be taken during the installation procedure to not damage the geotextile or drainage core in any way.

A vibrating type compactor with a properly shaped shoe sufficient to operate within the trench limits shall be used during each backfill lift placement.

Fittings for the geocomposite edge drain system shall conform to the manufacturer's specifications. Fittings shall be constructed to maintain integrity of the system under rigors of installation and long term expected earth loads, train loads and traffic loads, and shall be constructed with smooth inverts to provide efficient and unrestricted outflow.

Couplings used to join rolls of drain shall meet the specifications of the manufacturer. The coupling shall provide positive connection that is sufficient to prevent joint pull apart prior to and during the backfill operation.

The spacing of outlet fittings shall be as shown on the plans or as determined by the engineer.

The diameter of the outlet pipe shall be 4" non-perforated plastic pipe with a minimum pipe stiffness of 150 psi per ASTM D2412, (Schedule 40 PVC pipe in accordance with ASTM D1785, SDR 23.5 plastic pipe in accordance with ASTM D3034 or D2751). The solvent cement for the outlet pipe and fittings shall be in accordance with ASTM D2564, D2235 or D3138. The material composition of the outlet fittings shall be compatible for direct solvent welding to the plastic outlet pipe.

All connectors between drain and fittings/couplings shall be made soil tight as recommended by the manufacturer.

10.4.7 Measurement and Payment

Geocomposite edge drain systems shall be measured by the lineal foot.

Geocomposite drainage systems shall be paid by the lineal foot of product installed and accepted by the engineer, unless otherwise prescribed by the contract.

Proposed 1996 Manual Revisions to Chapter 2—Track Measuring Systems

Part 2—Track Measuring Vehicles

Page 2-2-7. Add the following new Article 2.3, as follows:

2.3 RECOMMENDED PRACTICE CONDITIONS FOR GAGE RESTRAINT MEASUREMENT

2.3.1 Definitions

The following definitions apply to the systems which have been employed to measure track gage strength.

AAR TRACK LOADING VEHICLE (AAR TLV): A specific track loading vehicle and instrumentation system owned by the Association of American Railroads Research and Test Department. AAR TLV is designed for research purposes to apply a wide range of hydraulically-generated computer-controlled vertical and lateral test loads to the track structure through an independent wheelset load bogie. When configured to perform gage restraint testing, AAR TLV uses AAR TLV loading conditions described below.

AAR TLV LOADING CONDITIONS: AAR TLV has typically measured gage restraint by applying continuous nominal vertical test loads of 33,000 lbs. and associated lateral loads of 18,000 lbs. The load values can be adjusted at any time.

FRA GAGE RESTRAINT MEASUREMENT SYSTEM (GRMS): A specific lateral restraint measurement system owned by the Federal Railroad Administration Office of Research and Development designed to perform continuous gage restraint measurement using mechanical wheels to measure unloaded and loaded gage and to apply test loads using FRA GRMS loading conditions described below. Nominal vertical test loads are controlled through test vehicle weight; lateral test loads are produced by a nominal constant-pressure hydraulic system acting through a split-axle wheelset operating as one of two wheelsets in a freight car truck assembly.

FRA GRMS LOADING CONDITIONS: Vertical load on each rail between 14,000 and 19,000 lbs. per wheel of test load axle. Corresponding simultaneous lateral load acting outward between rails of 10,000 and 18,000 lbs.

FRA GAGE WIDENING RATIO: Amount of observed gage widening in inches linearly normalized for 16,000 lbs. of applied lateral load, measured under FRA GRMS loading conditions.

FRA PROJECTED LOADED GAGE-24 (PLG24): The loaded gage estimated to occur by nonlinearly extrapolating measured FRA GRMS values for unloaded gage, loaded gage, and applied loads to a severe loading condition which is known to occur in revenue service as follows:

Vertical load on high rail of 33,000 [lbs]
Lateral load on high rail of 24,000 [lbs]

Vertical load on low rail of 33,000 [lbs]
Lateral load on low rail of 16,000 [lbs]

GAGE RESTRAINT MEASUREMENT SYSTEM (GRMS): Any track measurement system which applies suitable test loads and measures unloaded and loaded gages to measure the lateral restraint capacity of the track.

GAGE WIDENING RATIO (GWR): The measured difference between loaded and unloaded gage measurements, normalized to a specified amount of applied lateral load.

L/V RATIO: Numerical ratio of lateral load applied at a point on the rail to the vertical load applied at that same point.

LOAD SEVERITY: The net amount of lateral load applied to the fastener system after friction between rail and tie is overcome by any applied gage-widening lateral load.

LOADED GAGE: A measure of gage under specified vertical and lateral loading conditions.

LOADED GAGE UNDER TRAFFIC: The maximum gage which normally occurs at a location in track under the loads applied by the revenue service traffic on that track, as shown by free play and/or indications of motion between rail, fasteners, and ties or other track support.

PROJECTED LOADED GAGE (PLG): An extrapolated value for loaded gage calculated from actual measured loads and deflections. PLG is intended to represent the loaded gage measurement which would occur under hypothetical and typically more severe loading conditions.

NEUTRAL LOADING CONDITION: A lateral load of less than 250 lbs. applied to each rail with a vertical load of less than 300 lbs. applied to each rail.

TRACK LOADING DEVICE: A device which applies controlled and measured test loads to the track structure. Test loads may be applied laterally, vertically, or longitudinally.

TRACK LOADING VEHICLE (TLV): Any vehicle capable of operating on track and intended to apply test loads to the track structure.

UNLOADED GAGE: A measure of gage under neutral loading condition. Measurements must be made no closer than 10 feet from the lead axle of any truck or the load application axle, and no closer than 5 feet from the trailing axle of any truck.

2.3.2 Background

Gage restraint measurement is accomplished by making two gage measurements at the location in track where the gage restraint is to be measured. The first gage measurement is made with no significant loads applied. The second gage measurement is made with significant vertical loads applied to each rail and a significant gage-spreading lateral load applied between the rails.

The change in gage between the unloaded state and the loaded state indicates the gage restraint strength of the track. A large change in gage caused by a small lateral force indicates a track with weak lateral restraint; a small change in gage with a large lateral force applied indicates a location with strong lateral restraint. The overall measurement process has been adapted to operate on a continuous basis.

2.3.3 Considerations for Performing Lateral Restraint Measurements

2.3.3.1 Measurement System Design Considerations

The unloaded gage measurement can be made with either mechanical contact systems or with non-contact sensors of various types. The unloaded gage measurement must be made in an area of the track which is essentially free of other vehicle-imposed loads in order to assure a truly unloaded measurement. The operation of the overall measurement system must assure that each unloaded measurement is made to correspond with a loaded measurement taken at essentially the same point in the track.

The test loads to accomplish the loaded gage measurement are typically applied through flanged wheels on a telescoping axle. The inertial forces associated with rapidly moving the load applying components limit the ability of these systems to operate at high speeds. Existing systems have operated successfully in the 30 mph range.

The loaded gage is determined either with non-contact sensors or by measuring the distance which the axle telescopes to keep the flanges tight against the rail. The loaded gage measurement must be made at a point as near as possible to the point of load application to assure that measured deflections can be most accurately attributed to the measured applied loads. Each gage reading and the vertical and lateral forces at the loaded point are measured and recorded.

The corresponding unloaded and loaded gage measurements must be performed in a consistent manner so that factors such as rail lipping do not introduce error between the unloaded and loaded gage measurements.

The amount the rail head deflects as a result of the test loads depends on how much force is applied, how strong or weak the rail restraint condition is, and how much vertical and lateral load is being applied to the surrounding tie/fastener system. For the safety of test operations, test loads should not continue to be applied when loaded gage reaches 58 inches. Non-test vertical loads applied adjacent to the location under test will cause additional friction forces at the test point and will cause the measured lateral deflection to decrease. The presence of non-test variable vertical or lateral loads near the measurement point can be a significant source of error in the measurement process. The measurement process can be compensated for known adjacent vertical loads.

Any additional lateral load applied in the area surrounding the test location will cause the measured lateral deflection to change and can result in measurements which are not comparable with other measurement systems. Limitations must be placed on the distances, magnitudes, and variabilities of the adjacent non-test loads to make the deflection measurement consistent and repeatable.

2.3.3.2 Magnitude of Applied Test Loads

To be adequate for the purposes of determining gage restraint, the vertical test load must be sufficient to remove all free play between the base of the rail and the supporting tie plates, to properly represent the condition of the track under traffic conditions. The minimum recommended vertical wheel load for measurement is 10,000 pounds. This level will provide adequate seating of the rail. In addition to the need for adequate rail seating, when nominal vertical loads below this level are applied, dynamic variations in both lateral and vertical loading make it difficult to maintain a lateral load sufficient to assure that the minimum load severity of 3.0 kips is maintained without exceeding the 1.25 L/V derailment limit.

The lateral gage-spreading load must be sufficient to overcome all frictional resistance generated by the applied vertical loads and therefore to remove all lateral free play in the rail/fastener/tie plate/tie system, and to assure that some lateral force remains to apply a "proof test" load against the fastening system. The loading conditions recommended herein are based on a nominal rail to tie friction coefficient of 0.4.

If the applied lateral force is too large, damage to the tie and fastener system could result and/or derailment of the test vehicle can result. If the applied lateral force is too small, track strength will not be adequately tested and a misleading indication of strong track may result.

The relationships between applied vertical and lateral test loads which indicate the recommended range of loading regimes for continuous gage restraint testing are labelled Zone II in Figure 1. The loading regimes in Zone II provide sufficient vertical force to seat the rail and sufficient lateral force to assure that the fastener system is exercised, while limiting L/V ratios to acceptable levels and preventing permanent track damage due to excessive lateral forces.

The loading regimes contained in Zone IV have been found through field experience to be acceptable for continuous gage restraint testing. However, operations using load combinations in this zone must be conducted with caution, since L/V ratios exceed 0.8. Experience to date has shown that careful control of wheelset contour and lubrication of the wheel/rail contact point can permit successful testing in this zone.

The loading regimes in Zone III provide less than 3,000 pounds of lateral force in excess of the force reasonably likely to be needed to overcome friction between the tie plate and the tie surface,

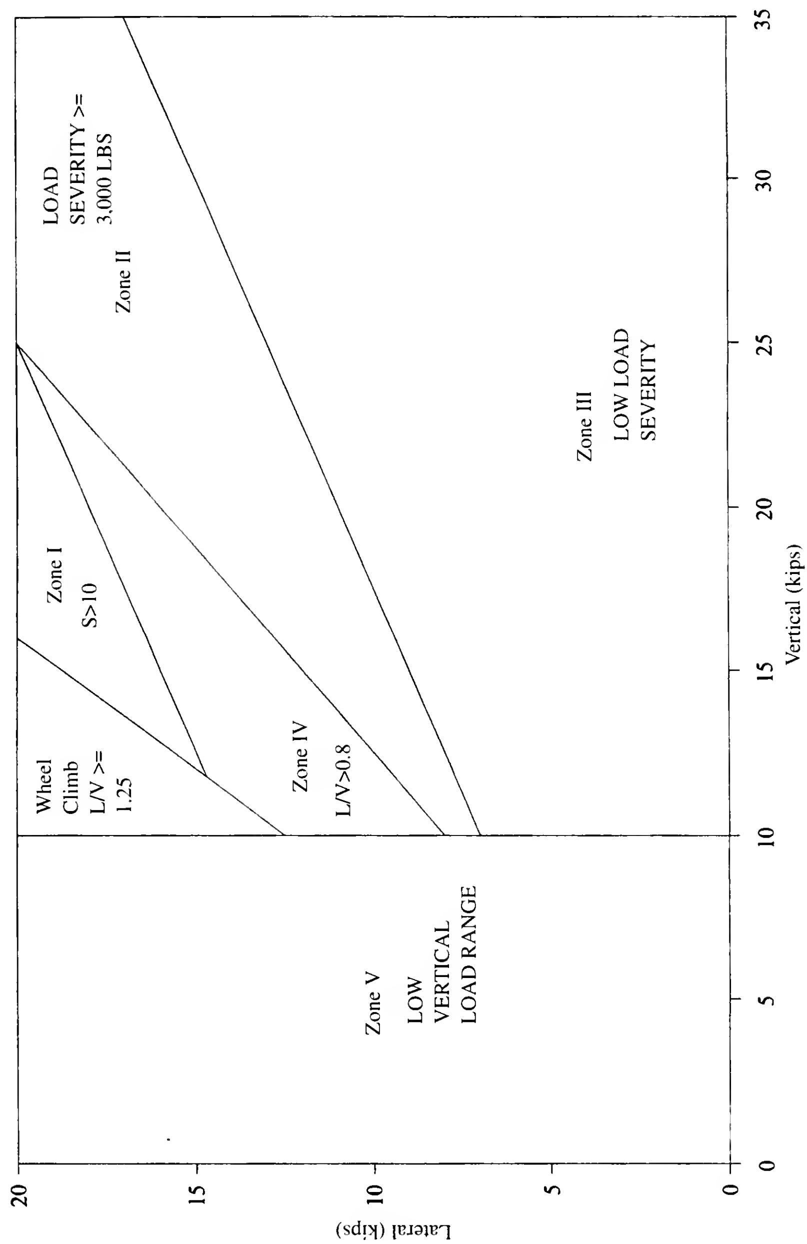


Figure 1. Gage Restraint Test Loads.

assuming a coefficient of friction at that interface of 0.4. Testing in Zone III may fail to detect weak locations in the track. The loading regimes in Zone I in general represent conditions in which 10,000 or more pounds of force in excess of friction are applied to the fasteners, and unrecoverable deflection may result. The small area at the left of Zone I and adjacent to Zone IV is a regime in which L/V ratios exceed 1.25. Testing in this area is not recommended due to unacceptable risk of wheel-climb derailment of the test wheelset.

The regimes in Zone V do not have the minimum required 10,000 lbs. vertical load. A minimum of 10,000 pounds of vertical load is recommended for the following reasons.

1. Analysis and experience has shown that a 10,000 pound vertical load is necessary to seat the rail and tie plate to reliably engage the fasteners. For Nominal vertical loads less than 10,000 lbs it becomes impractical to maintain the necessary lateral load without exceeding the 1.25 L/V ratio derailment limit
2. From a practical standpoint, when the nominal vertical load falls below 10,000 lb. the lateral and vertical loads must be prevented from varying more than 1,000 lbs. each to assure maintenance of a load severity above 3,000 pounds while not exceeding the derailment limit at an L/V of 1.25.

Gage restraint research and testing to date has clearly indicated that loaded track gage can change extremely rapidly from point to point, often within only one to three feet along the track. This means that any test system must have rapid response capacity to handle changes in load and rapid movements of the load applying system as it responds to rapidly changing track gage and track strength conditions, and to sustain substantial test loads despite the rapidly changing conditions in the track.

Test systems with inadequate mechanical capacity will produce erratic test loads. When such loads are too low, any extrapolation to higher test loads will be exaggerated and may result in false alarms for safety thresholds. Any system which overcompensates for low loads may inadvertently overload and damage the track as the system rapidly moves from an area of weak track into an area of stronger track. High capacity, very rapid response, and control of maximum force levels are essential for satisfactory gage restraint measurement.

The high variability of track gage and the rapid mechanical responses of the testing system mean that a rapid-response load measuring sensor system is also essential to assure accurate measurement of track gage strength. Experience has shown that systems which use averaging techniques or which do not measure at or near the point of load application may fail to record significant rapid changes in applied test loads. All load values stated in this recommended practice are instantaneous values applied and/or measured at the point and time of load application.

Experience has shown that a significant weak location in track may exist over an area of only three ties, and test systems may have difficulty in maintaining lateral load specifically at such weak locations which cause the test system to encounter rapidly widening gage. It is strongly recommended that any test system identify locations where insufficient test loads occur at any planned sample point, to avoid the possibility of systematically missing critical locations of track weakness in gage restraint.

The following specific practices are recommended to assure a consistent and repeatable gage strength measurement:

1. The unloaded gage measurement must be taken at a point at least 5 ft. (3.1 m) behind or 10 ft (6.2 m) in front of any load applying source greater than 350 lbs in either the lateral or vertical direction.
2. The applied vertical load must be at least 10,000 lbs. on each rail.

3. The loaded gage shall be measured within 1 foot (.31 m) of the load application point.
4. Application of gage widening loads must be immediately reduced or discontinued when loaded gage reaches or exceed 58 inches for the safety of test operations. The testing system shall identify and report any sampled location at which this occurs.
5. No vertical or lateral load shall be applied to the track for 10 feet leading the test load application point and for 5 feet trailing the test load application point.
6. The instantaneous applied loads must fall within the loading regimes indicated as Zone II on Figure 1, and may lie within Zone IV if operating precautions are taken to assure the safety of test operations. The testing system shall identify and report any sampled location where loads fall in Zone III or Zone V as a location at which the test was not valid due to inadequate lateral load to fully exercise the fastener system.
7. A continuous measurement requires that the track be sampled at an interval less than or equal to the distance between points providing gage restraint support, such as crossties or fastenings to a continuous track support medium. A sample at one foot intervals has been found acceptable on conventional track structures.

2.3.3.3 Issues Under Ongoing Study

The following issues are under continuing study: 1.) calibration requirements and procedures for measurement instrumentation; 2) recommended time intervals between inspections; and 3) recommending specific extrapolation relationships to anticipated gage readings at load levels higher than those actually measured. Thresholds which delimit safe or unsafe levels of gage restraint are beyond the scope of this recommended practice.

Proposed 1996 Manual Revisions to Chapter 4—Rail

Part 2—Specifications

Page 4-2-2-3. Article 5.1 Rail Tolerances

Delete present 5.1.4, width of either flange +/– 0.040

Add new 5.1.4, Asymmetry of head with respect to base +/– 0.060.

Add new 5.1.8, Flange height +0.025 –0.015

Page 4-2-3. Article 5.2. Replace present article with the following:

Verification of tolerances shall be made with the gages shown in Appendix B, or with other gages, as agreed by purchaser and manufacturer.

Page 4-2-6.20. Add the following new Appendix:

Appendix 3

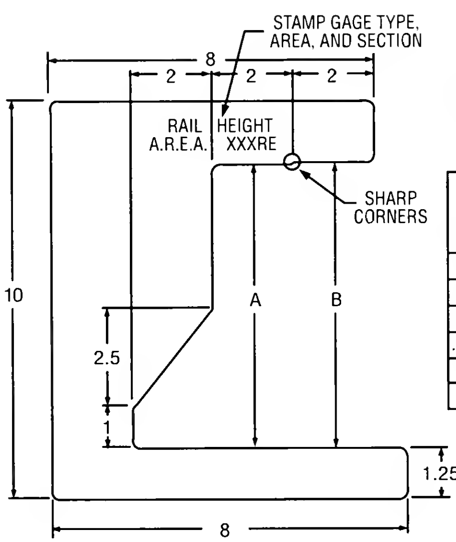
Drawings of gages for determining compliance with AREA rail section tolerances per Section 5.

Page 4-2-2 Section 3 Chemical Composition

Article 3.3. Revise the first sentence as follows: Separate analysis shall be made from test samples representing the front, middle (optional), and back of the heat preferably taken during pouring of the heat.

Article 3.5. Replace current article with the following:

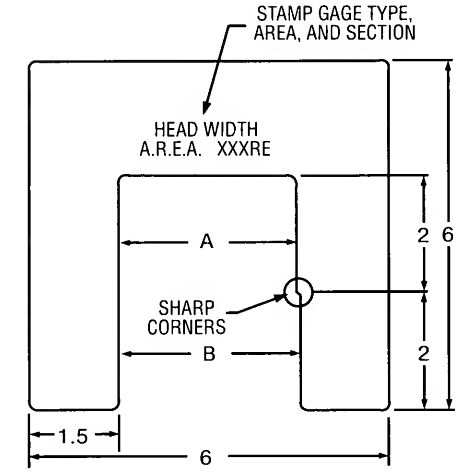
The analysis, most representative of the heat (clear of the transition zone for continuous cast steel), shall be recorded as the official heat analysis, but the purchaser shall have access to all chemical analysis determinations.



MAT'L: 12 GA NICKEL-PLATED SHEET STEEL OR STAINLESS. ALL CORNERS 0.125 RAD UNLESS NOTED.

AREA SECTION	DIMENSION (INCH) A $\binom{+0.002}{-0.000}$	DIMENSION (INCH) B $\binom{+0.000}{-0.002}$
115RE	6.610	6.665
119RE	6.797	6.852
132RE	7.110	7.165
133RE	7.047	7.102
136RE	7.297	7.352
140RE	7.297	7.352

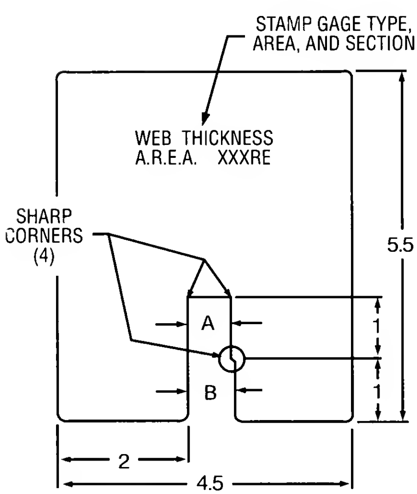
Proposed Gauge for Rail Height



MAT'L: 12 GA NICKEL-PLATED SHEET STEEL OR STAINLESS. ALL CORNERS 0.125 RAD EXCEPT AS NOTED.

AREA SECTION	DIMENSION (INCH)	
	A ($^{+0.002}_{-0.000}$)	B ($^{+0.000}_{-0.002}$)
115RE	2.689	2.749
119RE	2.626	2.686
132RE	2.970	3.030
133RE	2.970	3.030
136RE	2.908	2.968
140RE	2.970	3.030

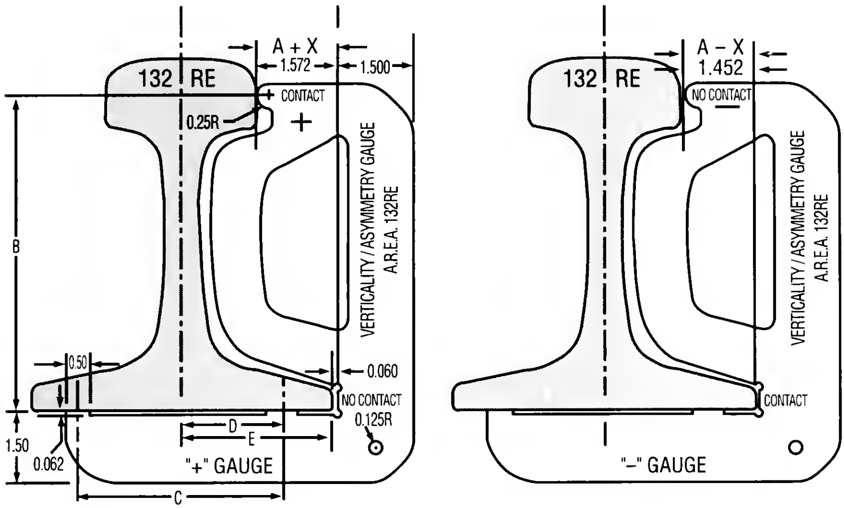
Proposed Gauge for Head Width



MAT'L: 12 GA NICKEL-PLATED SHEET STEEL OR STAINLESS. ALL CORNERS 0.125 RAD EXCEPT AS NOTED.

AREA SECTION	DIMENSION (INCH)	
	A ($^{+0.002}_{-0.000}$)	B ($^{+0.000}_{-0.002}$)
115RE	0.605	0.665
119RE	0.605	0.665
132RE	0.636	0.696
133RE	0.668	0.728
136RE	0.668	0.728
140RE	0.730	0.790

Proposed Gauge for Web Thickness

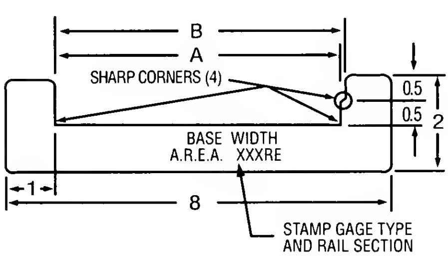


	115 RE	119 RE	132 RE	133 RE	136 RE	140 RE
A	1.404	1.437	1.512	1.545	1.546	1.554
B	5.875	6.063	6.375	6.313	6.563	6.563
C	3.661	3.661	4.016	4.016	4.016	4.016
D	1.890	1.890	2.008	2.008	2.008	2.008
E	2.750	2.750	3.000	3.000	3.000	3.000
X	0.060	0.060	0.060	0.060	0.060	0.060

NOTE: ALL DIMENSIONS IN INCHES

MATERIAL: 12 GA SHEET STEEL NICKEL PLATED OR STAINLESS

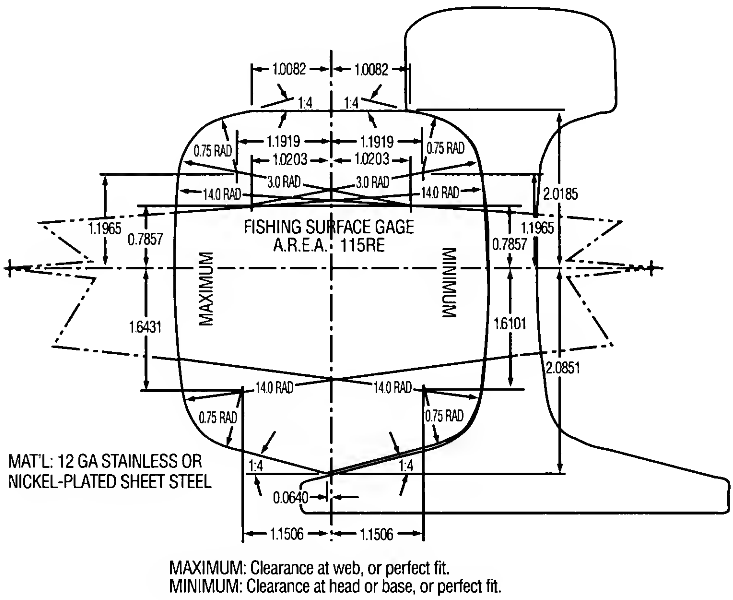
Proposed Gauge for Verticality/Asymmetry



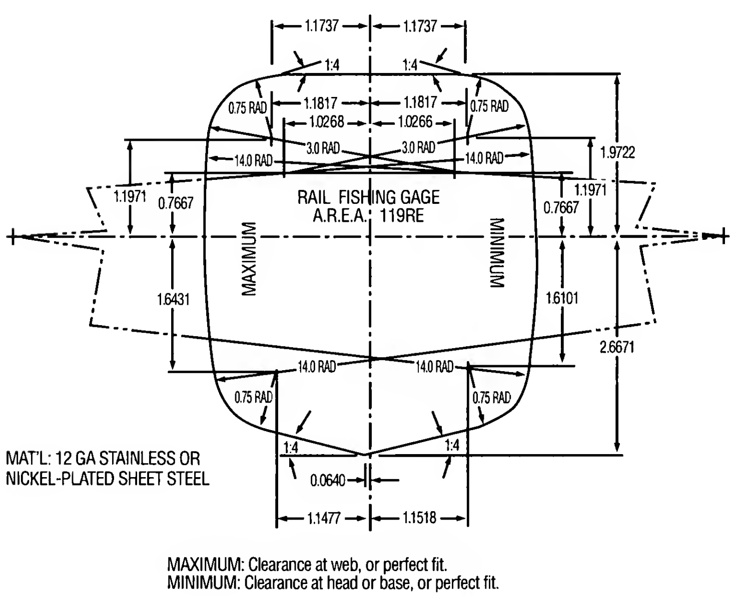
MAT'L: 12 GA NICKEL-PLATED SHEET STEEL OR STAINLESS.
FILLET OUTSIDE CORNERS 0.125 RAD EXCEPT AS SHOWN.

AREA SECTION	DIMENSION (INCH)	
	A $\left(\begin{smallmatrix}+0.002\\-0.000\end{smallmatrix}\right)$	B $\left(\begin{smallmatrix}+0.000\\-0.002\end{smallmatrix}\right)$
115RE	5.450	5.550
119RE	5.450	5.550
132RE	5.950	6.050
133RE	5.950	6.050
136RE	5.950	6.050
140RE	5.950	6.050

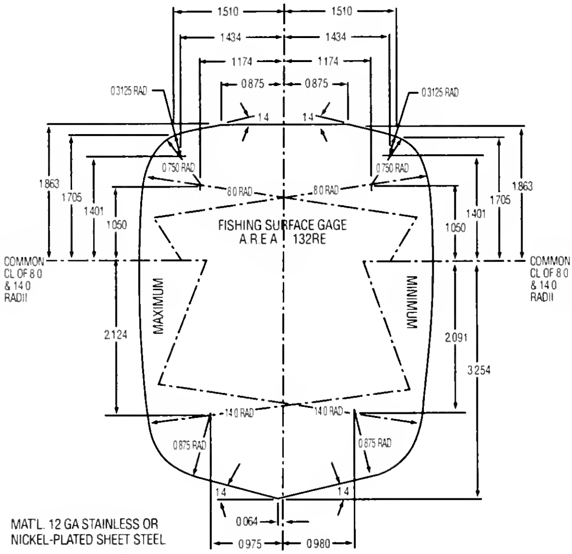
Proposed Gauge for Base Width



Proposed Fishing Surface Gage

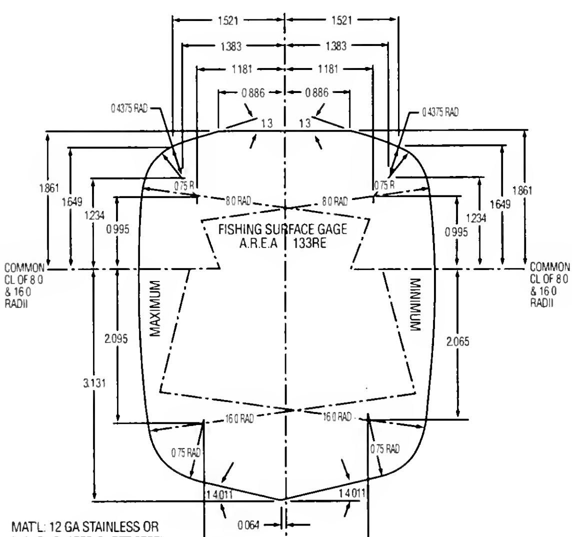


Proposed Fishing Surface Gage



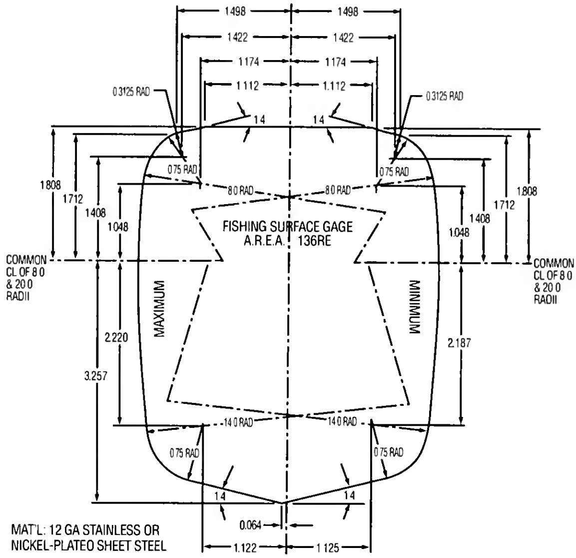
MAXIMUM: Clearance at web, or perfect fit.
MINIMUM: Clearance at head or base, or perfect fit.

Proposed Fishing Surface Gage



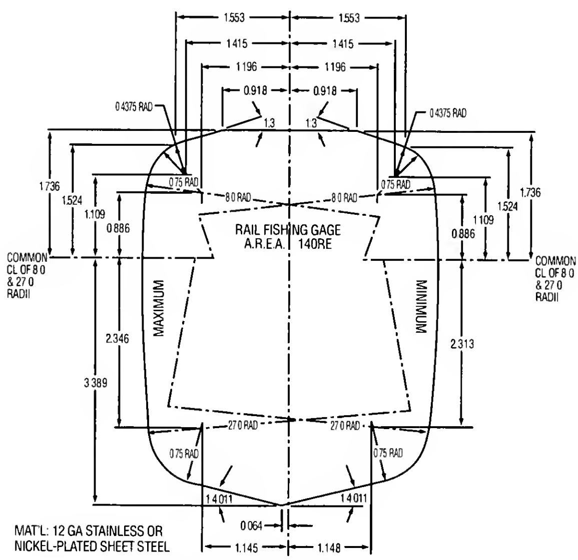
MAXIMUM. Clearance at web, or perfect fit.
MINIMUM. Clearance at head or base, or perfect fit.

Proposed Fishing Surface Gage



MAXIMUM: Clearance at web, or perfect fit.
MINIMUM: Clearance at head or base, or perfect fit.

Proposed Fishing Surface Gage



MAXIMUM: Clearance at web, or perfect fit.
MINIMUM: Clearance at head or base, or perfect fit.

Proposed Fishing Surface Gage

Add the following new specifications:

Specifications for Bonded Insulated Rail Joints

1. SCOPE

These specifications cover the design, materials, fabrication and qualification testing of bonded insulated rail joints.

2. ENGINEERING DRAWINGS

The manufacturer shall submit to the purchaser, for approval, drawings showing the material description, dimensions, fabrication tolerances and assembly methods where required.

3. INSPECTION

The purchaser's authorized representatives shall have free entry to the manufacturer's plant to inspect the processing and testing of all bonded insulated joints and/or their components. The manufacturer shall provide test specimens to satisfy the purchaser that the bonded insulated joints and/or their components are being supplied in accordance with this specification. All required qualification tests and production inspections shall be made prior to shipment unless otherwise stated by purchaser.

The manufacturer shall provide the purchaser with necessary copies of his quality assurance manual, for the purchaser's review and approval. Upon request, the manufacturer shall provide the purchaser with access to documentation of the active use and findings of the quality assurance procedures.

4. MATERIALS

4.1 General

All bonded insulated joints and/or components shall be new and conform to the requirements specified herein unless otherwise specified by the purchaser. All materials shall conform to the dimensional requirements for the rail section specified by the purchaser.

4.2 Full Contact Joint Bars

Joint bars for bonded insulated joints shall conform to the configuration of the rail section specified by the purchaser with allowances being made for the insulating material to be used and shall be fabricated from material which meets or exceeds the mechanical properties and workmanship requirements of the current A.R.E.A. "Specifications for Quenched Carbon Steel Joint Bars, Micro-alloyed Joint Bars and Forged Compromise Joint Bars" except as noted below. The fishing height of the joint bar with insulation shall be controlled to within +0 inch to −1/32 inch of the rail section specified. The contact surface of the joint bars adjacent to the rail shall be smooth and straight within a tolerance of ±1/32 inch using a 36 inch straight edge. No branding or other raised surfaces shall be permitted on the contact surfaces. All holes shall be deburred and conform to the size, tolerances and locations specified by the purchaser. Alternate metallic or non-metallic joint bars shall be used if specified by the purchaser.

4.3 Rail

When prefabricated bonded insulated joints are ordered, and rail is furnished by the manufacturer, the rail used in fabricating the bonded insulated joints shall conform to the chemical composition, mechanical properties, and workmanship requirements of the current A.R.E.A. Section 4.2 "Specifications for Steel Rails". The use of high strength rails for bonded insulated joints is recommended. The rail shall be saw cut with a variation in end squareness of not more than 1/32 inch. The lengths and drilling of each rail shall be as specified by the purchaser. All burrs from sawing and drilling shall be removed. To the extent possible, adjacent sawed ends of the rail shall be jointed by the bonded insulated joint bars. All raised letters, numerals, etc., within the joint area shall be removed, by grinding, to conform to the existing rail section prior to joint assembly. Should standard rail be utilized, end hardening is recommended and shall be in accordance with the current A.R.E.A. Specification for Steel Rails, Part 2—Supplementary Requirement S1—End Hardening; or, an alternate method if approved by the purchaser.

4.4 Insulating Materials

All insulation materials shall have electrical characteristics such that completed joints will meet or exceed the dielectric requirements of the AAR Signal Manual, Part 14.5.1, Paragraph D, and the Electrical Tests specified in Paragraph 7.3 herein. End post size shall be as specified by the purchaser with a thickness tolerance of +1/32 inch.

4.5 Fasteners

The bonded insulated joint shall be designed to be joined together with an adhesive and bolted together with six high strength bolts of a diameter to be specified by the purchaser. Every other bolt shall be reversed with the nut or fastener on the opposite side of the rail. The bolts, nuts, and washers, if required, shall conform to the chemical and mechanical requirements of ASTM Specification A90, or A325, as applicable, and have Class 2A and 2B thread fits. An alternate equivalent fastening system shall be used if specified by the purchaser.

4.6 Adhesive

The structural adhesive used as the bonding agent shall produce a minimum lap shear strength of 3500 psi at 75°F as per test prescribed in ASTM D-1002 (metal to metal). Adhesive shall be capable of meeting the above requirements for a period of one year from date of manufacture when stored as specified by the manufacturer. A corrosion inhibitor shall be included in the adhesive formulation.

5. WORKMANSHIP

5.1 General

The glue-bonded insulated joint is an assembly of insulating materials, steel and adhesive. Its design is for these dissimilar materials to perform as a homogeneous product. To accomplish this, care must be taken to ensure that quality control procedures are used and that no voids exist in the joint area.

5.2 Contact Surfaces

The steel contact surfaces of the bars and rail shall be cleaned to bright metal by an approved method, such as sand blasting. All grit and other residues must be removed from the steel contact surfaces to be bonded.

5.3 Adhesive

Enough adhesive must be used to completely cover the entire contact surfaces of the joint bars and rail and allow some excess adhesive to be squeezed out along the entire perimeter of the joint, when the joint is assembled. Any excess adhesive should be dressed around the perimeter and used to cover the bolt heads and nuts. The assembled joint shall be cured in accordance with the manufacturer's recommendations.

5.4 Rail Ends and Bolt Holes

The rail ends shall be saw cut with a variation in end squareness of not more than 1/32 inch. Sharp edges and burrs shall be removed by grinding. The bolt holes shall be free of sharp edges, burrs, loose scale, shavings and other foreign matter.

5.5 Fastener Torque

Fasteners must be tightened to required torque, following manufacturer's suggested sequence and procedures.

6. DIMENSIONAL TOLERANCE

6.1 Overall Straightness

Assembled joints shall not deviate from a straight line by more than the tolerances provided in Table 1. The deviation from a straight line must be reasonably uniform. Kinks are unacceptable except as provided in section 6.2.

Table 1.

Length of Rail and Joint	**13' to 39'**	**Over 39' to 60'**
Maximum mid-ordinate from a straight line for either side sweep or upsweep	1/4"	1/2"

6.2 Joint Area

The vertical alignment of the assembled joint shall be level, within a tolerance of +0.030" crown, as measured with a 36 inch straight edge. Dip shall not be permitted. See Figures 1 and 2.

The horizontal alignment of the assembled joint shall be straight, within a tolerance of 0.040 inch as measured with a 36 inch straight edge. See Figure 3.

Vertical offset between the two rail ends shall not exceed 0.030". Horizontal offset (kink) shall not exceed 0.020".

7. QUALIFICATION TESTING

7.1 General

Three bonded insulated joints shall be tested by the material components manufacturer as follows: Two bonded insulated joints shall be tested as specified in Section 7.2, Longitudinal Compression Test and the remaining bonded insulated joint shall be tested first in accordance with Section 7.3, Electrical Resistance Test, then subjected to a test as specified in Section 7.4, Rolling Load Test. After completion of the rolling load test, the joint shall be resubjected to the electrical resistance test.

Qualification testing shall not commence until the engineering drawings are approved by the purchaser. For each design and/or material change, the material components manufacturer shall be required to perform these qualification tests only on a one-time basis unless otherwise agreed upon by both the manufacturer and the purchaser.

If the bonded insulated joint being purchased has been previously qualified, the manufacturer shall provide access to the test results to subsequent purchasers. If the manufacturer makes any changes in the materials or the design, the manufacturer shall requalify the new joint through the testing prescribed herein before production is resumed.

7.2 Longitudinal Compression Test

Two bonded insulated joints shall be completely assembled per manufacturer's recommendations. Two pieces of rail of the prescribed rail section, each two feet long shall be utilized for each joint. Each joint assembly shall then be sawed in half where the rails are butted together. The sawing

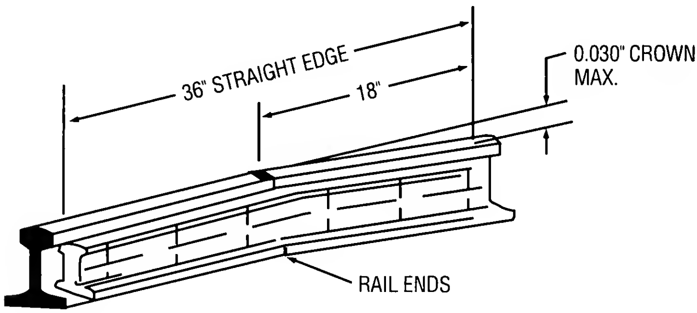


Figure 1. Elevation of Joint Showing Misalignment Tolerance in Vertical Alignment per Section 6.2.

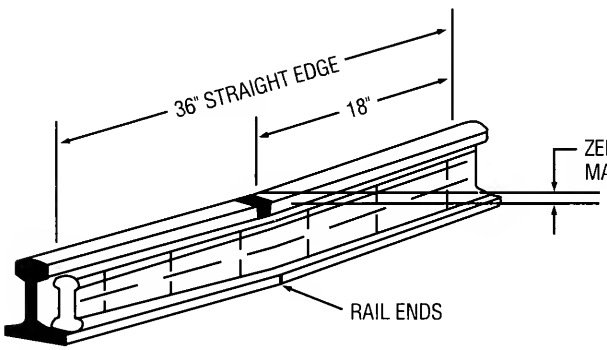


Figure 2. Elevation of Joint Showing Misalignment Tolerance in Vertical Alignment per Section 6.2.

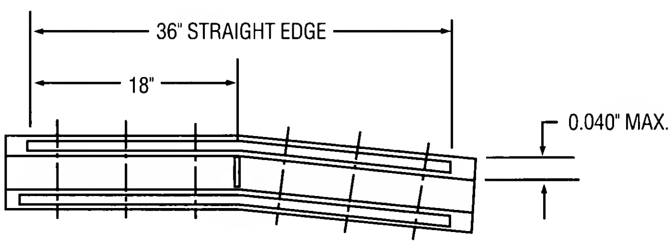


Figure 3. Plan View of Joint Showing Misalignment Tolerance in Horizontal Alignment per Section 6.2.

Tolerances for Inspection of Bonded Insulated Joints

shall be done in a manner which will prevent overheating and damage to the bonding agent, and the cut shall be perpendicular to the center line of the top of the rail with a tolerance of plus or minus one degree. The sawn ends of the bars at one end of the test piece, and the end of rail at the other, shall have fair bearing in the test machine to ensure that the loading and reaction are through the centroid of the rail, and parallel to its axis.

7.2.1

Load shall be applied parallel to the running surface of the rail in increments of 25,000 pounds. Each load increment shall be maintained constant until the longitudinal deflection of the rail ceases before increasing the load by the next increment.

The load shall be increased in these increments until a total load of 650,000 lbs. is attained for rail weights of 132 lb. or greater, or failure occurs. For rails less than 132 lb., a total load of 600,000 lbs. shall be used. At each increment of loading, the load and differential movement of the rail and joint bars, measured to 0.001 inch, shall be recorded. If an alternate method of performing this test is used, it shall be submitted to the purchaser for prior approval.

7.2.2

The acceptance criterion for the longitudinal compression test shall be as follows: At no time shall any of the bonded insulated joints show any indication of slippage during or before the total prescribed load for the rail section involved is applied to the joint. At the completion of the test, after the load on the rail has been released, the relative position of the rail and joint bar shall be within 0.020 inch of its original value.

7.3 Electrical Resistance Test

7.3.1

A rail joint shall be fully assembled in accordance with manufacturer's recommendations on two lengths of rail for an electrical resistance test. The dry rail and joint assembly shall be supported on dry non-electrical conducting material.

7.3.2

Apply 2200 volts, 60 Hz, A.C., rail to rail which shall be held for a duration of not less than five seconds.

The acceptance criterion shall be that there shall be no flashover or puncture through the insulation which is evident by failure to maintain the voltage through the time stipulated.

7.3.3

Apply 500 volts, D.C. rail to rail and each rail to one bar, each test for a duration of five (5) seconds according to either of the following:

$$R\,(\text{ohms}) = \frac{500\,(\text{volts})}{I\,(\text{amps})} \quad \text{where 1 megohm} = 1{,}000{,}000 \text{ ohms}$$

Method 1: Measure the actual current flow (I) through the joint to the nearest 0.1 microampere and record. Calculate the resistance (R) using the formula

or, Method 2: Use a megohmmeter that reads directly in megohms (resistance).

The acceptance criterion for these tests shall be a minimum resistance of ten (10) megohms.

7.4 Rolling Load Test

7.4.1

The bonded joint shall be mounted on a 33 inch stroke rolling load test machine and supported on 36 inch centers with the joint centered between supports.

7.4.2

A wheel load of 44,000 lbs. shall be applied to the rail. The stroke shall have a range of 33 inches, centered as shown on Figure 4. The load on the rail shall be applied for 2,000,000 cycles and the deflection of the rail at the center line of rail shall be measured and recorded when the wheel load is over both points A and B for every 500,000 cycles and recorded to the nearest 0.001 inch.

7.4.3

Alternative method of testing the joint dynamically may be submitted to the purchaser for approval.

7.4.4 Acceptance Criteria

At all times the deflection of the bonded insulated joint shall not exceed 0.065 inches.

8. ACCEPTANCE

To be accepted, all prefabricated bonded insulated joints and bonded insulated joint materials must fulfill all of the requirements of this specification.

9. PACKAGING AND HANDLING

9.1

The proposed method of packaging, handling and loading for all items shall be submitted to the purchaser for approval before production is begun.

9.2

Prefabricated bonded insulated joints shall be handled and loaded in a manner that will not damage the insulated joint or the rail.

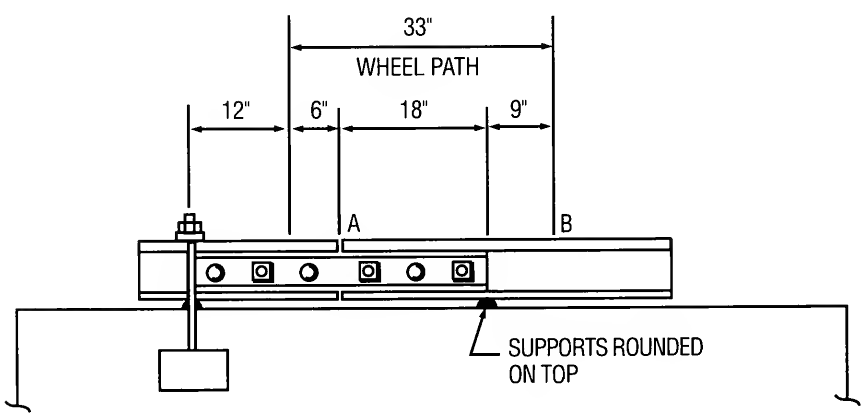


Figure 4.

10. MARKING

Date of manufacture, name of manufacturer, rail section and metallurgy shall be marked on the joint such that it will remain during the life of the joint.

Rail shall be marked with paint as to length of finished plug, and color coded as to metallurgy. Colors to be as agreed upon between purchaser and manufacturer.

Add the following new specifications:

Specifications for Non-Bonded Encapsulated Insulated Rail Joints

1. SCOPE

These specifications cover the design, materials, fabrication, and qualification testing of non-bonded encapsulated insulated rail joints for current AREA rail sections.

2. ENGINEERING DRAWINGS

The manufacturer shall submit to the purchaser, for approval, drawings showing the material description, dimensions, fabrication tolerances and assembly methods where required.

3. INSPECTION

The purchaser's authorized representatives shall have free entry to the manufacturer's plant to inspect the processing and testing of all non-bonded encapsulated insulated joints and/or their components. The manufacturer shall provide test specimens to satisfy the purchaser that the non-bonded encapsulated insulated joints and/or their components are being supplied in accordance with this specification. All required qualification tests and production inspections shall be made prior to shipment unless otherwise stated by purchaser.

The manufacturer will provide the purchaser with necessary copies of his quality assurance manual, for the purchaser's review and approval. Upon request, the manufacturer will provide the purchaser with access to documentation of the active use and findings of the quality assurance procedures.

4. MATERIALS

4.1 General

All encapsulated insulated joints and/or components shall be new and conform to the requirements specified herein unless otherwise specified by the purchaser. All materials shall conform to the dimensional requirements of the rail section specified by the purchaser.

4.2 Core Bars

Core bars shall be fabricated from material that meets or exceeds the mechanical properties and workmanship requirements of the current AREA Specifications for Quenched Carbon-Steel Joint Bars, Alloyed Steel Bars and Forged Compromise Joint Bars, except as noted below. Alternate types of core bars may be used if approved by the purchaser.

4.3 Tolerances for Finished Bars

The fishing height of the joint bar with insulation shall be controlled within +0 inch to −1/32 inch of the rail section specified. The contact surface of the joint bars adjacent to the rail shall be smooth and straight to within ±1/32 inch on the horizontal plane using a 36 inch straight edge. Any variation from a straight line in the vertical plane shall be to make the joint bars high in the center by up to 1/32 inch maximum. No branding or other raised surfaces shall be permitted on the contact surfaces. All bolt holes shall conform to location specified by the purchaser. Bolt hole tolerances shall be to AREA plan 1010-89, Rail End and Joint Drilling.

4.4 Insulating Materials

All insulation materials shall have electrical characteristics such that completed joints will meet or exceed the dielectric requirements of the AAR Signal Manual, Part 14.5.1, Paragraph D, and the electrical tests specified in Paragraph 6.2 herein. End post size shall be as specified by the purchaser with a thickness tolerance of 1/32 inch.

4.5 Fasteners

The encapsulated insulated joint shall be designed to be bolted together with heat treated oval neck track bolts of a diameter to be specified by the purchaser. Washer plates shall permit every other bolt to be reversed with the nut or fastening on the opposite side of the rail, unless otherwise specified by the purchaser. The nuts, bolts and lock washers shall conform to AREA design requirements and to the AREA Chemical and Mechanical Specifications for Heat-Treated Carbon-Steel Track Bolts and Carbon Steel Nuts, unless otherwise specified.

5. WORKMANSHIP

5.1 General

The encapsulated insulated joint is an assembly of insulating materials and steel. Its design is for those dissimilar materials to perform as a homogeneous product. To accomplish this, care must be taken that quality control measures are used.

5.2 Surface Preparation of Core Bars

Surface preparation shall be such as to promote optimum adhesion of the encapsulation to the core bars. A primer may be used to promote adhesion.

6. QUALIFICATION TESTING

6.1 General

Two encapsulated insulated joints shall be tested by the material components manufacturer as follows: one encapsulated insulated joint shall be tested first in accordance with Section 6.2, Electrical Resistance Test, then subjected to a test as specified in Section 6.3, Rolling Load Test. After completion of the rolling load test, the joint shall be resubjected to the electrical resistance test. The remaining insulated joint shall be submitted to slow bend test as specified in section 6.4.

6.2 Electrical Resistance Test

6.2.1

A rail joint shall be fully assembled in accordance with manufacturer's recommendations on two lengths of rail for an electrical resistance test. The dry rail and joint assembly shall be supported on dry non-electrical conducting material.

6.2.2

Apply 500 volts D.C. rail to rail. Each test will be for a minimum duration of five seconds and there shall be a minimum resistance of 10 megohms.

6.2.3

Apply 1500 volts A.C. rail to rail. Each test will be for a duration of not less than three seconds without flashover or puncture between all metallic parts and other metallic parts insulated therefrom.

6.3 Rolling Load Test

6.3.1

The encapsulated joint shall be assembled on two full section rails of the specified section, with fastenings as specified by the purchaser, and the bolts torqued to the purchaser's standard specified torque. The resulting assembly shall be mounted on a 33 inch stroke rolling load test machine and supported on 36 inch centers with the load centered between supports.

6.3.2

A wheel load of 44,000 pounds shall be applied to the rail. The stroke shall have a range of 33 inches, centered as shown on Figure 5. The load on the rail shall be applied for 2,000,000 cycles and the deflection at the center line of the rail shall be measured and recorded when the wheel load is over points A and B for every 500,000 cycles and recorded to the nearest 0.001 inch.

6.3.3

Alternative ways of testing the joint dynamically may be submitted to the purchaser for approval.

6.3.4 Acceptance Criteria

On completion of rolling load test the joint shall show no evidence of material failure. Maximum acceptable deflection shall be as agreed upon between the purchaser and the manufacturer.

6.4 Slow Bend Test

6.4.1 Applicability

A slow bend test is useful for evaluating the overall strength and stiffness of joints for rails in modern heavy sections such as those shown in Part 1 of this Chapter, when they are required to carry trains at high speed, or must carry substantial tonnages of heavy cars at other than low speed.

In general the determination of whether a product such as encapsulated insulated joint bars of a particular type will give satisfactory performance in track can only be made by in-track experience; but the slow bend test is useful, in particular, in determining whether the structural properties of the core bars are adequate.

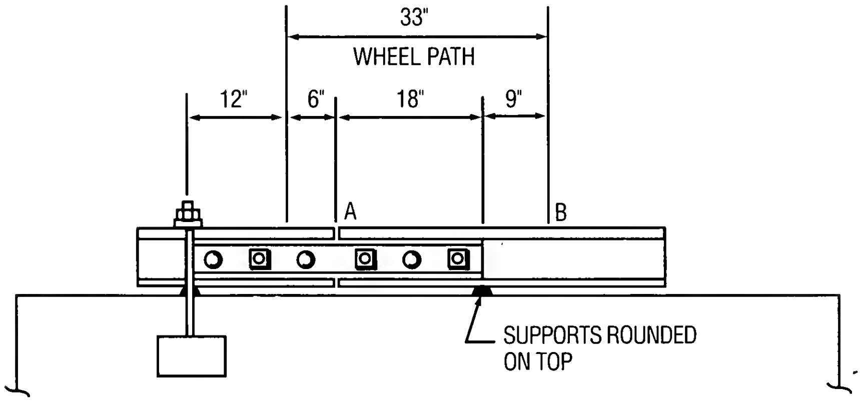


Figure 5.

6.4.2 Test Method

Standard test method used with 132/136 lb. encapsulated insulated joints is shown in Appendix 1. In cases where the purchaser buys more 132/136 lb. bars than any other, he may choose to accept or reject all sections of bars based on results from the 136 lb. section. If the purchaser does not use 132/136 lb. bars, acceptance/rejection criteria shall be as agreed between the supplier and purchaser.

6.4.3 Bending Strength

When tested in standard slow bend test machine using method shown in Appendix 1, no damage or permanent deflection shall appear in 132/136 lb. bars applied to full section 136 lb. rail under 50 kips of vertical loading, or under 12 kips of horizontal loading.

6.4.4 Stiffness

At maximum vertical loading of 50 kips elastic deflection of the rail joint assembled as per section 6.4.3 shall not exceed 0.8 inches in the vertical direction. At maximum lateral loading of 12 kips, elastic deflection of the rail joint so assembled shall not exceed 0.7 inches in the lateral direction.

6.4.5 Causes for Rejection

Besides failure to meet any of the criteria given in sections 6.4.3 and 6.4.4, any breakage, cracking, splintering, bulging, delamination or visible permanent kinking of the joint, or any obvious kink or change of slope of the load/deflection curve, will be considered evidence of damage and will be cause for rejection.

7. ACCEPTANCE

To be accepted, all encapsulated joints and components thereof must fulfill all requirements of this specification.

8. PACKAGING AND HANDLING

Packaging shall be done on the basis of one kit per carton, and shall be in accordance with the manufacturer's standard packaging and handling methods, unless otherwise specified by the purchaser.

9. MARKING

Month and year of manufacture, name of manufacturer and rail section or sections fitted shall be marked on encapsulated insulated joint bars so it will remain during the life of the joint.

Appendix 1. Method of Slow Bend Test

Test shall be run on new joints of size and type prescribed by the manufacturer for use on 136 RE rail, using bolts specified by the manufacturer.

Joint(s) shall be assembled on two sections of new 136 RE rail to current AREA specification, in accordance with manufacturer's plans and directions. Bolts shall be tightened to torque prescribed by the manufacturer. Huck bolts or other connecting devices not capable of being re-tightened after application shall not be used with non-bonded encapsulated insulated rail joints.

Rail shall be supported on 72 inch span, with joint centered between supports, and central static loading applied. For vertical load tests, dial gauges shall record vertical deflection at points located 3 inches on either side of the central loading point and on the center of the rail base. For lateral loading tests, load shall be applied at center of span through rail neutral axis, and deflections at rail head and at edge of rail base shall be measured by dial gauges located 3 inches on either side of loading point.

After taking initial dial gauge readings, vertical load shall be applied in increments of 5 kips and deflections on the two dial gauges shall be recorded for each increment. The average of the two deflection readings shall be plotted against load to produce a vertical load/deflection curve.

After taking initial readings, lateral load shall be applied in increments of one kip and deflections on the four dial gauges shall be recorded for each load increment. The average of the four deflection readings shall be plotted against load to produce a lateral load/deflection curve.

Vertical and lateral load tests shall be conducted separately; i.e., vertical and lateral loads shall not be applied to the joint at the same time during the test.

Some nonlinearity of the load-deflection curve may be observed under the initial loading cycle, due to initial set of the plastic encapsulation material and bedding-in of the joint against the rail. When agreed between manufacturer and purchaser, retest may be made by cycling the joint up to full specified vertical and lateral load five times, retightening the bolts to specified torque and doing the prescribed vertical and lateral load-deflection tests over again.

Miscellaneous Part

Delete pages 4-M-5 and 4-M-6, "Specifications for Heat-Treated Carbon Steel Tee Rails (USS CURVEMASTER) as Produced by the United States Steel Corporation."

Proposed 1996 Manual Revisions to Chapter 5—Track

Part 3—Curves

Page 5-3-13. Vertical Curves, replace the current article with the following revision:

Vertical Curves

1. Vertical curves as calculated in item 6 below should be used to connect all changes in gradients.

2. The length of vertical curve is determined by changes in gradient, vertical acceleration and the speed of the train.

3. The purpose of the vertical curve is to ease the change of gradients in order to reduce coupler and diaphragm binding and eliminate the danger of breaking trains in two as a direct result of train action. In addition, the proper vertical curve will provide for passenger comfort on passenger trains.

4. A vertical curve which is concave upwards shall be denoted as a sag. A vertical curve which is concave downwards shall be denoted as a summit.

5. The vertical curve may be either circular or parabolic in shape.

6. The minimum length of the vertical curve for both sags and summits is determined by the following formula (except that in no case should the length of vertical curve be less than 50 feet long):

$$L = \frac{D \times V^2 \times K}{A}$$

Where:

L = Length of vertical curve in feet.

D = Absolute value of the algebraic difference in rates of grades (expressed as a percentage).

V = Speed of the train in miles per hour.

K = 0.0215 which is a conversion factor to give L in feet.

A = Vertical acceleration in feet/sec/sec (ft/sec^2).

7. The recommended vertical acceleration is 0.60 feet/sec/sec (0.02 g) for both sags and summits.

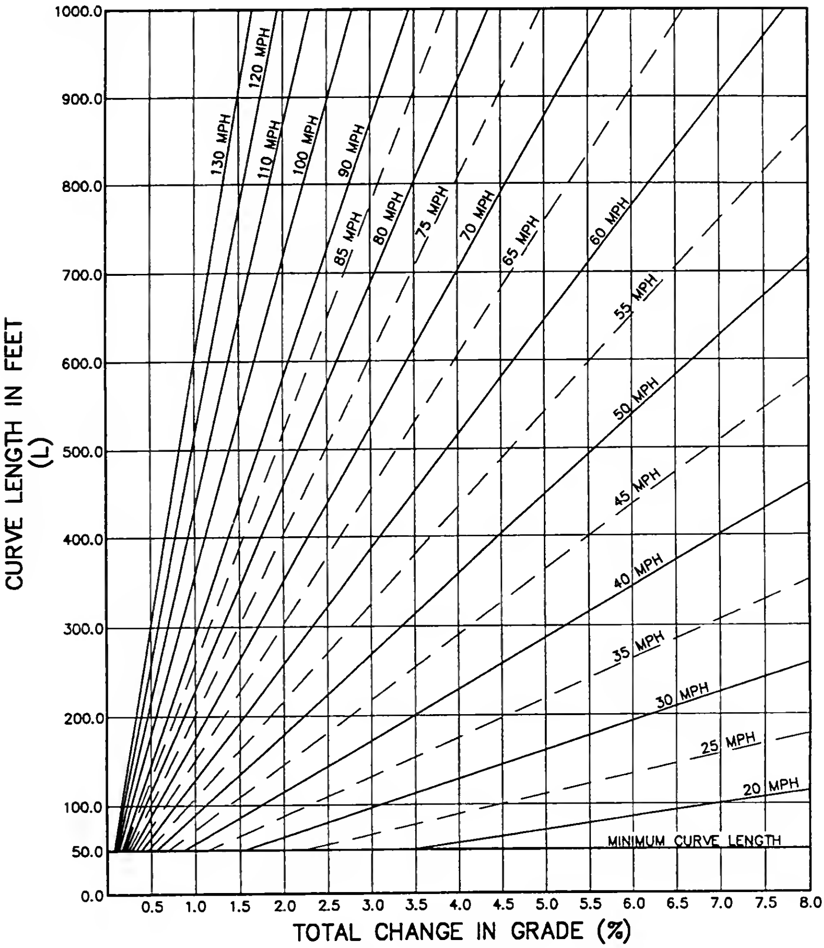
8. The recommended minimum distance between vertical curves shall not be less than 100 ft.

9. The train speed to be used in the above formula for establishing the length of vertical curve should be the maximum speed found on that particular subdivision or route. Special attention should be paid to locations where local conditions have dictated a speed restriction now in place, but where such a restriction might be removed at a later date.

10. It is not recommended to place special trackwork within the limits of a vertical curve.

11. Curves constructed to this formula should not present any problems for the current generation of equipment. Slow speed curves, such as hump crests, should, however, be designed with consideration for vertical clearance rather than using this formula.

NOTE: Values for various speeds and change in gradients have been graphed for reference.



FORMULA: $L = (D \times V^2 \times K)/A$

WHERE: V = Speed of the train in miles per hour.
L = Length of vertical curve in feet.
D = Absolute value of the algebraic difference in rates of grades expressed as a percentage.
A = 0.6 feet/sec/sec.
K = 0.0215 = conversion factor for $(MPH)^2$.

Recommended Minimum Lengths for Vertical Curves

Page 5-13-14. Add the following new article on Compensated Gradients:

Standard Compensated Gradients

1. Compensation of gradients due to the effects of horizontal curvature is recommended on all gradients, but is essential on ruling gradients.

2. The purpose of the compensated gradient is to aid uphill trains on grades by equating the total resistance of a train on a horizontal curve on a gradient to that of the total resistance of a train on tangent track on a gradient.

3. The amount of gradient compensation is determined by the compensation factor and the degree of curvature.

4. The recommended compensation factor to be used for standard gauge track is 0.04 percent per degree of curve. This corresponds to the resistance created by standard three piece trucks on non-lubricated curves. If radial trucks or self steering trucks are to be used, the compensation factor can be decreased.

5. The recommended compensated gradient due to curvature shall be calculated as follows:

$$G_c = G - 0.04D$$

Where:

G = gradient before compensation, expressed in percent.

D = degree of curvature expressed in decimals of degrees.

G_c = compensated gradient expressed in percent.

6. For further information, Chapter 16 should be consulted.

Proposed 1996 Manual Revisions to Chapter 8—Concrete Structures and Foundations

Part 1—Materials, Tests and Construction Requirements

Page 8-1-22. Section 1.10 Joints. Replace current section with following revised section on concrete jointing:

1.10 CONCRETE JOINTING

1.10.1 Scope

(a) This recommended practice is applicable to the design of concrete slabs and walls in all types of concrete structures i.e., bridges, buildings, flat work, etc.

1.10.2 Types of Jointing

(a) Expansion joints are filled separations between adjoining parts of the concrete structure which are provided to allow for relative movements, such as those caused by thermal changes.

(b) Contraction joints are sawed, tooled, or inserted in a concrete surface to create a weakened plane and control the location of cracking resulting from dimensional changes associated with shrinkage.

(c) Construction joints occur where two successive placements of concrete meet, across which it is desirable to maintain bond between the two concrete placements, and through which any reinforcement which may be present is not interrupted .

1.10.3 Expansion Joints

(a) Expansion joints allow for differential movement of the concrete mass in all directions, on either side of the joint. These may also be referred to as isolation joints.

(b) Expansion joints shall be installed as shown on the plans or as specified by the Engineer.

(c) Finger joints and other mechanical systems are not included in these recommended practices.

(d) Jointing materials shall be in accordance with ASTM D 994 or ASTM D 1751. There shall be no connection across the joint by welded wire fabric or structured steel. See Figure 1.10.3.

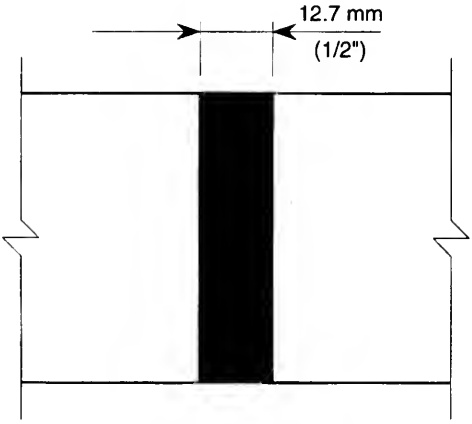


Figure 1.10.3. Full Depth Expansion Joint.

1.10.4 Expansion Joint in Walls

(a) Expansion joints between the finished surface and the waterstop shall be filled with a material such as 12.7 mm (½") thick strip of Premoulded Expansion Joint meeting ASTM D 994 or ASTM D 1751.

1.10.5 Contraction Joints

(a) Contraction joints allow for differential movement across the joint only in one direction, usually in the plane of the finished surface. They are provided to allow for drying shrinkage of the concrete.

(b) Contraction joints in slabs-on-grade shall be located and detailed as shown on the plans. Unless otherwise shown or noted, joints shall be spaced at 4.6–7.6m (15 to 25 foot) intervals in each direction.

(c) Contraction joints for slabs-on-grade shall be made by one of the methods shown in Figure 1.10.5 or as shown on the plans.

(d) Sawing of contraction joints shall be done as soon as the concrete has hardened sufficiently to prevent aggregates being dislocated by the saw, and shall be completed before shrinkage stresses become sufficient to produce cracking. Sawing shall not be done when the concrete temperature is falling, unless approved by the Engineer.

(e) Contraction joints may also be accomplished by means or methods specifically designed to create a plane of weakness in freshly placed concrete.

(f) Contraction joints may also be made by other methods if approved by the Engineer. Contraction joints shall be cleaned and filled with polymeric sealant conforming to ASTM D 1190 or ASTM D 3405 or as specified by the Engineer.

(g) Prior to the application of a polymeric joint sealing material, a heat resistant backer rod should be inserted to a minimum depth of 12.7 mm (½") below the slab surface. The remaining reservoir should then be filled flush with the slab surface (See Figure 1.10.5).

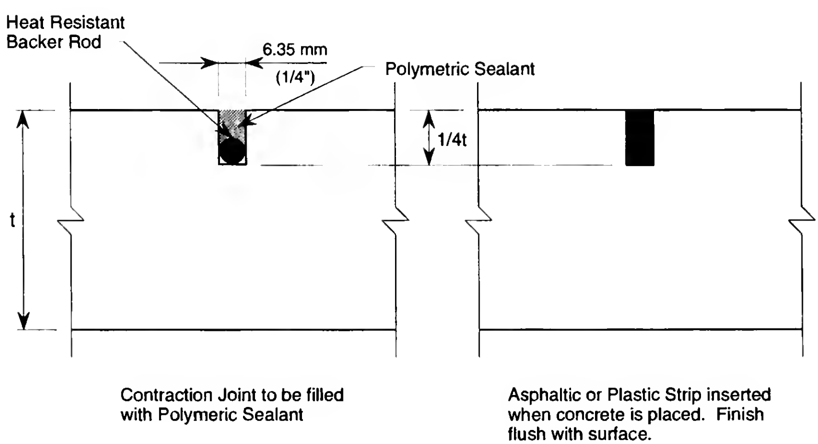


Figure 1.10.5.

1.10.6 Construction Joints

(a) Construction joints allow for no differential movement across the plane of the joint. They are provided only at locations where casting is temporarily suspended or interrupted.

(b) The procedures specified in Article 1.13.9 for bonding fresh concrete to hardened concrete shall be followed in the formation of all construction joints. The reinforcement shall continue through the joint. For concrete without reinforcement, sharing strengths shall be provided by means of keys or dowel bars as the Engineer may require.

(c) Structures or portions of the structures shall be continuously cast except as herein modified. When necessary to provide construction joints not indicated or specified by contract documents, such construction joints shall be located as approved by the Engineer and formed so as not to impair the strength, appearance, or durability of the structure.

1.10.7. Watertight Construction Joints

(a) Construction joints shall not be used in watertight construction unless shown on the plans or approved by the Engineer.

(b) Where a construction joint is used in water tight construction, special care shall be taken in finishing the concrete to which the succeeding concrete is to be bonded. The consistency of the concrete shall be carefully controlled and the surface shall be protected from loss of moisture as described in Article 1.17.4.

(c) Where construction joints are required to be watertight, a continuous keyway shall be constructed in the interface of the first section of concrete placed with an approved waterstop embedded in this first placement. One half of the waterstop is embedded in the first placement and the remaining material shall be embedded in the adjacent placement. See Figure 1.10.7 for details. The concrete shall be thoroughly vibrated to ensure uniform contact over the entire surface of the waterstop. The waterstop shall be in conformance with Corps of Engineers Specification CRD C 572 (PVC) or CRD C 513 (Rubber).

NOTE: Keyed joints should not be used in slabs less than 6 inches (152.4 mm) thick.

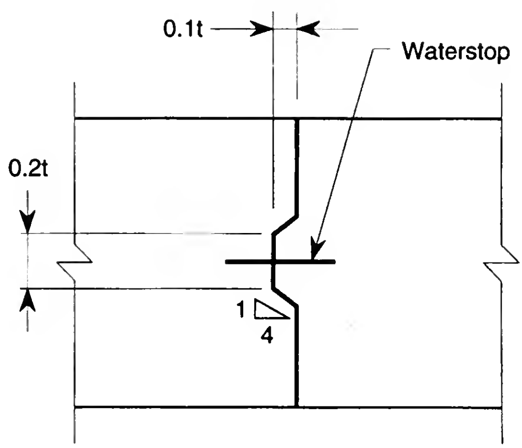


Figure 1.10.7. Keyed Construction Joint with Waterstop Inserted Perpendicular to the Plane of the Joint.

Add new Part 26 on Design of Segmental Bridges, as follows:

Part 26—Recommendations for the Design of Segmental Bridges

26.1 GENERAL REQUIREMENTS AND MATERIAL

26.1.1 General

(a) The specifications of Part 26 are intended for design of longitudinally and/or transversely post-tensioned bridges utilizing normal weight concrete constructed with either precast or cast-in-place box segments of single or multiple cells, or combinations thereof, as well as simple span and continuous segmental beam-type bridges. The specifications pertain to bridges of all sizes and are not restricted to bridge span lengths of 500 feet or less. Unless otherwise stated or superceded by these specifications, the provisions of the AREA Manual for Railway Engineering are intended to apply to the design of segmental concrete bridges.

26.1.2 Notations

Notations are in accordance with Part 2 and Part 17 and the following:

A = area of concrete surrounding a bar, (see Article 26.15.2) sq. in.

A_b = bearing area of tendon anchorage, sq. in.

A'_b = maximum area of the portion of the concrete anchorage surface that is geometrically similar to and concentric with the bearing area of the tendon anchorage, sq. in.

A_{cc} = area of concrete in compression chord, sq. in.

A_{cn} = area of one face of a truss node region, sq. in.

A_{cp} = area enclosed by outside perimeter of concrete cross section, sq. in.

A_{cs} = area of inclined compression strut, sq. in.

A_g = gross area of concrete cross section, sq. in.

A_l = total area of additional longitudinal reinforcement to resist torsion, sq. in.

A_o = area enclosed by shear flow path, See Section 26.8.2.10, sq. in.

A_s = area of nonprestressed tensile reinforcement, sq. in.

A'_s = area of compression reinforcement, sq. in.

A^*_s = area of prestressed reinforcement in tension zone, sq. in.

A_t = area of one leg of continuous, closed transverse torsion reinforcement within a distance s, sq. in.

A_V = area of transverse shear reinforcement within a distance s, sq. in.

a = portion of single span, end span, or span adjacent cantilever arm subject to shear lag effects (see Figure 26.2.3.2A), ft.

b = top or bottom flange width either side of web (see Figure 26.2.3.2C), ft.

b_e = minimum effective shear flow web or flange width to resist torsional stresses, [see Paragraphs 26.8.2(j), 26.8.2(e) and 26.8.3 (a)], ft.

b_f = effective flange width coefficient for interior portion of span (see Figures 26.2.3.2A and 26.2.3.2B), unitless.

b_m = effective width of flange (see Figure 26.2.3.2A), ft.

b_{m1} = effective width of cantilever flange of box girder (see Figure 26.2.3.2C), ft.

b_{m2} = effective width of half of interior top flange of box girder (see Figure 26.2.3.2C), ft.

b_{m3} = effective width of half of bottom flange of box girder (see Figure 26.2.3.2C), ft.

b_{mf} = effective width for center portion of span (see Figures 26.2.3.2A and 26.2.3.2B), ft.

b_{ms} = effective width at support or for cantilever arm (see Figures 26.2.3.2A and 26.2.3.2B), ft.

b_n = effective flange width for lateral distribution of post-tensioning force (see Figure 26.2.3.2D), ft.

b_{no} = web width at anchorage of post-tensioning force (see Figure 26.2.3.2D), ft.

b_o = web width (see Figure 26.2.3.2C), ft.

b_s = effective top or bottom flange width coefficient at supports and for cantilever arms (see Figures 26.2.3.2A and 26.2.3.2B), ft.

b_w = minimum web width, [see Paragraph 26.8.2 (e)], in.

b_1 = width of cantilever flange of box girder (see Figure 26.2.3.2C), ft.

b_2 = width of half of interior top flange of box girder (see Figure 26.2.3.2C), ft.

b_3 = width of half of interior bottom flange of box girder (see Figure 26.2.3.2C), ft.

CE = weight of specialized construction equipment, kips.

CLE = longitudinal construction equipment load, kips.

CLL = construction live load, psf, normally taken as 10 psf.

c = portion of continuous span adjacent to interior support subject to shear lag effects (see Figure 26.2.3.2A), ft.

D = sum of dead load of structure (DL), superimposed dead load (SDL), and permanent effects of erection loads (EL), kips.

DIFF = differential (unbalanced) dead load from one cantilever, kips.

DL = dead load of structure only, kips.

DT = thermal differential from centerline of typ flange to centerline of bottom slab, degrees F.

d = distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement, in. For prestressed concrete members, the greater of the distance from the extreme compression fiber to the centroid of the prestressed tension reinforcement or 0.8h may be used, ft.

d_a = depth of anchor plate, in.

d_c = thickness of cover from tension fiber to center of bar, (see Article 26.15.2) in.

d_o = total depth of section (see Figure 26.2.3.2A), ft.

d_{ON} = construction height of secondary beam (see Figure 26.14A), ft.

d_{OH} = construction height of main beam (see Figure 26.14A), ft.

d_{sp} = total depth of symmetric concrete prism above and below the anchor plate (also assumed to be the length of the anchorage zone), in.

E_{cm} = secant modulus of elasticity, psi or ksf.

E_{eff} = effective modulus of elasticity for long term loads considering creep deformations, psi or ksf.

EL = permanent effect of erection loads (final state), psi or ksf.

e = base of Naperian logarithms

F_{bst} = total bursting force (tensile) due to a tendon anchorage, kips.

F_r = radial force due to tendon curvature, lb. per ft.

f'_c = specified compressive strength of concrete, psi or ksf.

$\sqrt{f'_c}$= square root of specified compressive strength of concrete, (see Paragraph 26.8.2 (f) for limit) psi or ksf.

f'_{ci}= compressive strength of concrete at time of initial prestress, psi or ksf.

f_{cn} = compressive stress in the concrete node regions, (see Paragraph 26.8.4 (f)) psi or ksf.

f_{cp} = permissible concrete compressive stress under anchorage, psi or ksf.

f_{cu} = crushing strength of diagonally cracked concrete, (see Paragraph 26.8.4 (d)) psi or ksf.

f_{pc} = compressive strength in concrete after allowance for all prestress losses, psi or ksf. Critical stress to be determined at:

a) the centroid of the cross section resisting external loads, or

b) the junction of the web and compression flange when the centroid lies within the flange, or

c) in composite members, the stress at (a) or (b) for stresses due to both prestress and the moments resisted by the precast member acting alone.

f_s = stress in nonprestressed reinforcement under erection loads, (see Article 26.15.2), psi.

$f_{p\mu}$ = ultimate strength of prestressing steel, psi.

f'_s = stress in compression reinforcement, psi.

f_{si} = steel stress at beginning of time intervals t_i psi.

f^*_{su} = average stress in prestressed reinforcement at ultimate load, psi. f_{sy} = specified yield strength of nonprestressed reinforcement, psi.

f_{sy} = specified yield strength of nonprestressed reinforcement, psi.

f^*_y= yield point stress of prestressing steel, psi.

h = overall thickness of member, in.

IE = impact load from equipment

K = $\sqrt{[1 + fpc/2\sqrt{f'_c}]}$, factor for torsional cracking moment (see Sections 26.8.2.10 & 26.8.2.12)

l = span length, (see Figure 26.2.3.2A) ft.

l_1 = span length for use in determining effective flange width, (see Figure 26.2.3.2A) ft.

M_u = factored moment at section, in-lb or ft-lb.

N_{uc} = factored compressive axial force normal to cross section, lb.

N_{ut} = factored tensile axial force normal to cross section, lb.

P = tendon force, (see Articles 26.12.3 and 26.12.6.1) lb.

p_{cp} = outside perimeter of the concrete cross section, in.

P_j = tendon jacking force, kips

ph = perimeter of centerline of outermost continuous closed transverse reinforcement, in.

R = tendon radius of curvature, (Section 26.12.3) ft.

R = rib shortening and creep effects, (see Articles 26.4.2 and 26.4.4.1) kips.

R_{lr} = loss of prestress due to steel relaxation, low relaxation strand, psi.

R_{sr} = loss of prestress due to steel relaxation, stress relieved steel, psi.

S = shrinkage effects, (see Article 26.4.4.1) kips.

S_u = force in a truss member due to factored ultimate loads, lb.

SDL = superimposed dead load, kips.

s = spacing of shear or torsion reinforcement measured parallel to the longitudinal axis of the member, in.

s = bar spacing, in. (see Article 26.15.2)

T = sum of effects of thermal rise or fall (TRF) and thermal differential, (DT) kips.

TRF = thermal rise or fall, degrees F.

T_c = torsional cracking moment, (see Paragraph 26.8.2 (j)) in.-lb.

T_n = nominal torsion resistance, in-lb.

T_o = tendon stress at jacking end, psi.

T_u = factored torsion at section, in-lb.

U = load due to segment unbalance on opposite cantilever ends, kips.

V_c = nominal shear strength provided by concrete, lb.

V_n = nominal shear force resisted by member, lb.

V_p = component of the effective prestressing which acts in the direction of the applied shear. (see Paragraph 26.8.1 (g) and 26.8.1.5.2) lb.

V_s = nominal shear resisted by the 45° truss model as measured by the stirrup capacity, lb.

V_U = factored shear force at section, lb.

WTdl = area of concrete surrounding a bar, (see Article 26.15.2), sq. in.

WUP = wind uplift on cantilever, kips.

Z = correction dimension for location of center of gravity of tendon bundle in duct, (see Article 26.7.3) in.

Z = quantity for detailing of reinforcement to control flexural cracking during erection, (see Article 26.15.2) kips per inch.

σ_o = average compressive stress in the concrete section due to the post-tensioning anchorage force after the force is distributed over the depth, d, of the section, psi.

σ_y = transverse tensile stress in the concrete section due to the post-tensioning anchorage force, psi (see Figure C26.10.1).

ϕ = strength reduction factor (see Article 26.4.3).

ϕ_b = strength reduction factor for bearing (see Article 26.4.30).

ϕ_c = creep coefficient, ratio of creep strain to elastic strain.

ϕ_f = strength reduction factor for flexure (see Article 26.4.3).

ϕ_v = strength reduction factor for shear and diagonal tension (see Article 26.4.3).

μ = friction coefficient (per radian)

α = total angular deviation from jacking end to point x, radians.

26.1.3 Definitions

Definitions are in accordance with Part 2 & Part 17 and the following:

Anchorage Blister—Build-out in the web, flange, or web-flange junction to provide area for one or more tendon anchorages.

Closure—Cast-in-place concrete segment or segments used to complete a span.

Confinement Anchorage—Anchorage device for a post-tensioning tendon that functions on the basis of confinement of the concrete in the immediate anchorage zone by confinement reinforcing (stirrups, spirals or other devices to provide confinement to the concrete).

Deviation Saddle—Build-out in the web, flange, or web-flange junction to provide for change of direction of an external tendon.

External Tendon—Tendon located outside the flanges or webs of the structural member, generally inside the box girder cell.

General Bursting Forces—Bursting forces due to all of the tendons anchored at a cross section. Dependent on the overall concrete dimensions, and the magnitude, direction and location of the total prestressing force anchored.

General Zone—The region in front of the anchor which extends along the tendon axis for a distance equal to the overall depth of the member. The height of the general zone is taken as the overall depth of the member. In the case of intermediate anchorages which are not at the end of a member, the general zone shall be considered to also extend along the projection of the tendon axis for about the same distance before the anchor.

Internal Tendon—Tendon located within the flanges or webs (or both) of the structural member. All internal tendons shall be designed and constructed as bonded tendons.

Local Zone—The region immediately surrounding each anchorage device. It may be taken as a cylinder or prism with transverse dimensions approximately equal to the sum of the projected size of the bearing plate plus the manufacturer's specified minimum side or edge cover. The length of the local zone may also extend the length of the anchorage device plus an additional distance in front of the anchor equal to at least the maximum lateral dimension of the anchor.

Launching Bearing—Temporary bearings with low friction characteristics used for launching of bridges constructed by the incremental launching method.

Launching Nose—Temporary steel assembly attached to the front of an incrementally launched bridge to reduce superstructure moments during launching.

Low Relaxation Steel—Prestressing strand in which the steel relaxation losses have been substantially reduced by additional manufacturing procedures (stretching at elevated temperatures).

Secondary Moment—Restraint moments induced in continuous post-tensioned structures due to forces induced by the tendons at the time of stressing. The secondary moment changes with time only due to prestress losses.

Strut-and-Tie Model—A structural model used for analysis of shear, torsion and other forces based on a truss analysis by assuming compression struts in the concrete and tension ties in reinforcement.

Temperature Gradient—Variation of temperature of the concrete over the cross section.

Type A Joints—Cast-in-place concrete joints and wet concrete.

Type B Joints—Epoxied joints or dry joints between precast units.

26.1.4 Concrete

(a) Structural concrete used in segmental construction shall have a minimum 28-day strength of 4500 psi, or greater as specified by the Engineer. The required concrete strength at the time of stressing shall be determined in accordance with Article 26.5.2.

26.1.5 Reinforcement

26.1.5.1 Prestressing Steel

(a) As per Part 17.

26.1.5.2 Reinforcing Steel

(a) ASTM Grade 60 unless otherwise specified.

(b) All bridge deck reinforcement, including any reinforcement projecting from the web into the deck, shall be provided with a corrosion protective system in aggressive environments.

26.2 METHODS OF ANALYSIS

26.2.1 General

(a) Elastic analysis and beam theory may be used to determine design moments, shears, and deflections. The effects of creep, shrinkage, and temperature differentials shall be considered, as well as the effects of shear lag. Shear lag shall be considered in accordance with the provisions of Article 26.2.3.

26.2.2 Strut-and-Tie-Models

(a) Strut-and-tie-models may be used for analysis when tensile stresses exceed the tensile strength of the concrete, and for areas where strain distribution is non-linear.

26.2.3 Effective Flange Width

26.2.3.1 General

(a) Effective flange width may be determined by elastic analysis procedures,[5,6] by the provisions of Section 3-10.2 of the 1983 Ontario Highway Bridge Design Code[7] or by the provisions of Article 26.2.3.2.

26.2.3.2 Effective Flange Width for Analysis, and for Calculation of Section Capacity and Stresses

(a) Section properties for analysis and for calculation of the effects of bending moments and shear forces may be based on the flange widths specified in this section, or may be based on flange widths determined by other procedures listed in Article 26.2.3.1. The effects of unsymmetrical loading on effective flange width may be disregarded. For flange width, b, less than or equal to 0.3 d_o, b_m may be assumed equal to b, where d_o is taken as the web height in accordance with Figure 26.2.3.2C. For flange widths, b, greater than 0.3 d_o, the effective width may be determined in accordance with Figure 26.2.3.2A and 26.2.3.2B. The value of b_s shall be determined using the greater of the effective span lengths adjacent to the support. If b_{mf} is less than b_{ms} in a span, the pattern of the effective width within the span may be determined by the connecting line of the effective width b_{ms} at adjoining support points. However, b_m shall not be greater than b.

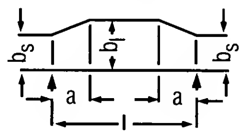
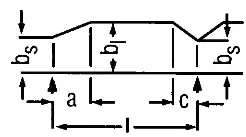
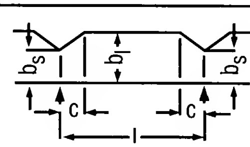
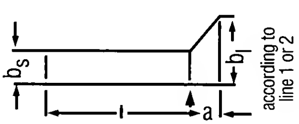
Line \ Column	1		2	3
	System		Pattern of $\frac{b_m}{b}$	
1	Single-span girder			$l_i = l$
2	Continuous girder	End span		$l_i = 0.8\,l$
3		Inner span		$l_i = 0.6\,l$
4	Cantilever arm			$l_i = 0.5\,l$
a = b, but not exceeding 0.25 l ; c = 0.1 l				

Figure 26.2.3.2A. Pattern of the effective flange width b_m.

(b) The section properties for normal forces may be based on the pattern according to Figure 26.2.3.2D, or may be determined by more rigorous analytical procedures.

(c) Stresses due to bending, shear and normal forces may be determined by using their corresponding section properties.

(d) For the superposition of the bending stresses of the main load-bearing structure over the slab bending stresses generated by local loads, the former may be assumed to have a straight line pattern in accordance with Figure 26.2.3.2C. The linear stress distribution is determined from the constant stress distribution under the condition that the flange force remains unchanged.

(e) The capacity of a cross-section at the ultimate state may be determined by considering the full flange width effective.

26.2.4 Transverse Analysis

(a) The transverse design of box girder segments for flexure shall consider the segment as a rigid box frame. Flanges shall be analyzed as variable depth sections considering the fillets between the flange and webs. Combinations of track loads, if the structure may support more than one track, shall be positioned to provide maximum moments, and elastic analysis shall be used to determine the

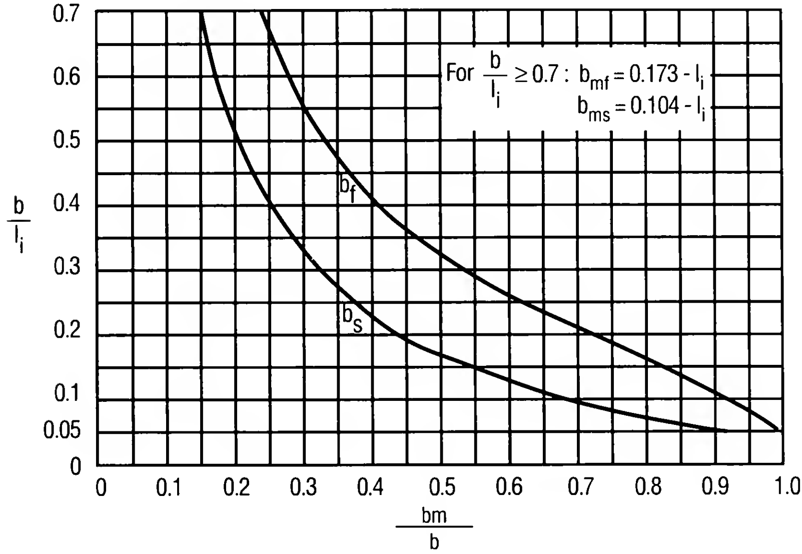


Figure 26.2.3.2B. Effective flange width $\frac{bm}{b}$ coefficients b_f, b_s.

effective longitudinal distribution of wheel loads for each load location. Tracks shall be positioned on the structure in accordance with clearance policies. Consideration shall be given to the increase in web shear and other effects on the cross-section resulting from eccentric loading or unsymmetrical structure geometry.

(b) Influence surfaces[10,11,12] or other elastic analysis procedures may be used to evaluate live load plus impact moment effects in the top flange of the box section.

(c) Transverse elastic and creep shortening due to prestressing and shrinkage shall be considered in the transverse analysis.

(d) The secondary effects due to prestressing shall be included in stress calculations at working load. In calculating ultimate strength moment and shear requirements, the secondary moments or shears induced by prestressing (with a load factor of 1.0) shall be added algebraically to the moments and shears due to factored ultimate dead and live loads.

26.2.5 Longitudinal Analysis

26.2.5.1 General

(a) Longitudinal analysis shall be in accordance with the provisions of Article 26.2.1. Longitudinal analysis of segmental concrete bridges shall consider a specific construction method and construction schedule, as well as the time-related effects of concrete creep, shrinkage, and prestress losses.

(b) The secondary effects due to prestressing shall be included in stress calculations at working load. In calculating ultimate moments and shear requirements, the secondary moments or shears induced by prestressing (with a load factor of 1.0) shall be added algebraically to moments and shears due to factored dead and live loads.

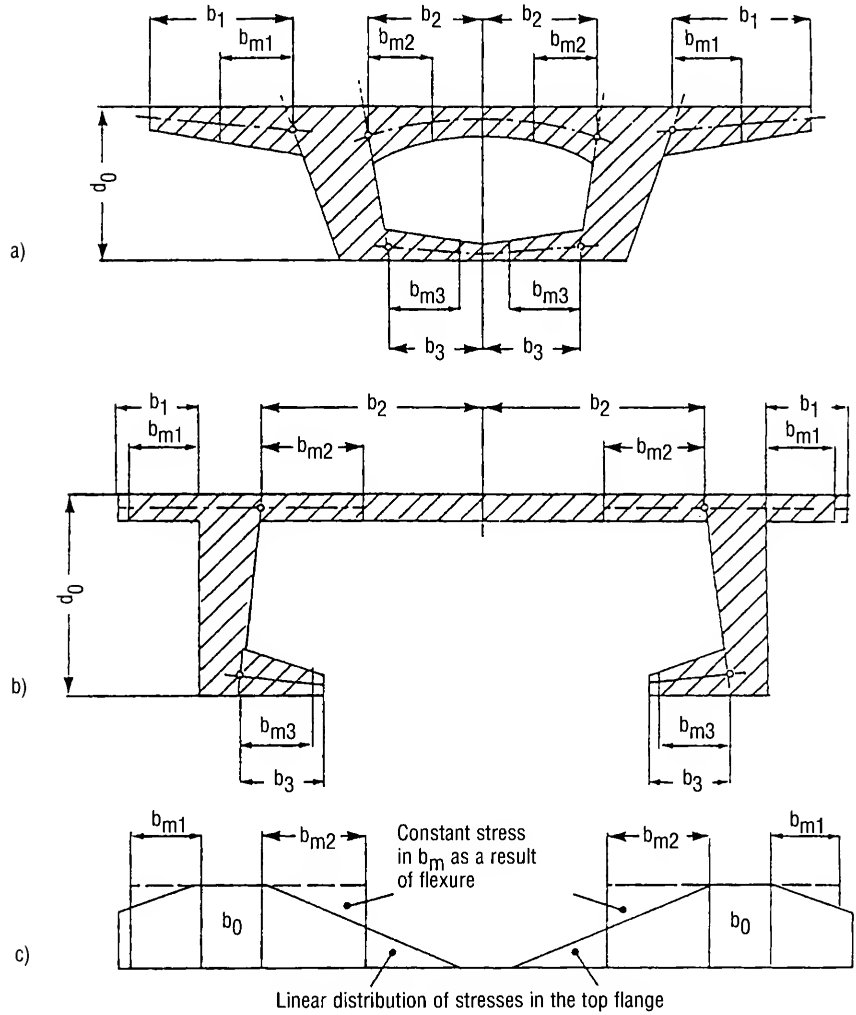


Figure 26.2.3.2C. Cross sections and corresponding effective flange widths, b_m, for bending and shear.

(c) Internal Tendons shall be designed and constructed as bonded tendons. Details of construction methods resulting in unbonded or partially unbonded internal tendons are not allowed.

26.2.5.2 Erection Analysis

(a) Analysis of the structure during the construction stage, shall consider the construction load combinations, stresses, and stability considerations outlined in Article 26.4.4.

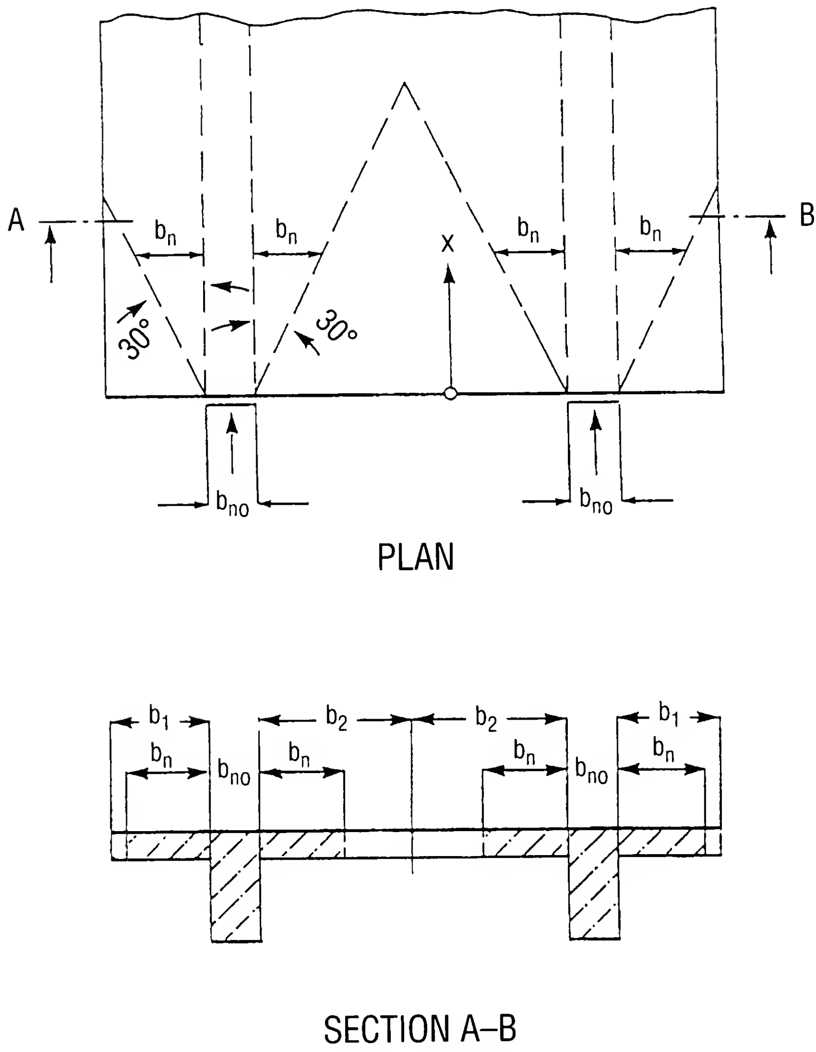


Figure 26.2.3.2D. Effective flange widths b_n for normal faces.

26.2.5.3 Analysis of the Final Structural System

(a) The final structural system shall be analyzed for redistribution of erection stage moments resulting from the effects of creep and shrinkage, and from any change in the statical system, including the closure of joints. Thermal effects on the final structural system shall be considered in accordance with Article 26.3.4. The effect of prestress losses occurring after closure shall be evaluated in accordance with Article 26.6. The maximum moments resulting from the above analyses shall be utilized in conjunction with the combinations of loads specified in Article 2.2.4 for determination of required flexural strength.

26.3 DESIGN LOADS

26.3.1 General

(a) All loadings shall be in accordance with the latest edition of the Manual For Railway Engineering except as provided below.

26.3.2 Dead Loads

(a) Unit weight of concrete (including reinforcing steel)—155 pcf or as determined for the project. Weight of diaphragms, anchor blocks, or any other deviations from the typical cross section shall be included in the dead load calculations.

26.3.3 Erection Loads

(a) Erection loads comprise all loadings arising from the designer's anticipated system of temporary supporting works and/or special erection equipment to be used in accordance with the assumed construction sequence and schedule. The assumed erection loads (magnitude and configuration) and acceptable closure forces due to misalignment corrections shall be stated on the drawings. Due allowance shall be made for all effects of any changes to the statical structural scheme during construction. The application, changes or removal of the assumed temporary supports or special equipment shall be considered by taking into account residual "built-in" forces, moments, deformations, secondary post-tensioning effects, creep, shrinkage and any other strain induced effects.

(b) All elements of the bridge shall be designed for the anticipated construction system assumed by the Engineer and shown on the plans. Any accepted contractor proposals which present differing construction loads shall be evaluated, by the Engineer, for effects upon the structure.

26.3.4 Thermal Effects

26.3.4.1 Normal Mean Temperature

(a) Unless more precise local data are available, normal mean temperature for the location shall be taken as the average of the January and July values from Figures 26.3.4.1A and 26.3.4.1B,[19] respectively.

26.3.4.2 Seasonal Variation

(a) For the purposes of design of the structure, the minimum and maximum over-all temperatures shall be taken from Figures 26.3.4.1A and 26.3.4.1B, respectively, unless more precise local data is available.

(b) The temperature setting variations for bearings and expansion joints shall be stated on the bridge plans.

26.3.4.3 Thermal Coefficient

(a) The coefficient of thermal expansion used to determine temperature effects shall be taken as 6.0 x 10-6 per degree Fahrenheit, unless more precise data are available.

26.3.4.4 Differential Temperature

(a) Positive and negative differential superstructure temperature gradients shall be considered in accordance with Appendix A of National Cooperative Highway Research Program Report 276 "Thermal Effects in Concrete Bridge Superstructures."[19] More precise data may be used if available.

26.3.5 Creep and Shrinkage

(a) Effects due to creep and shrinkage strains shall be calculated in accordance with provisions of Article 26.2.5.3. The creep coefficient ϕ_c may be evaluated in accordance with the provisions of

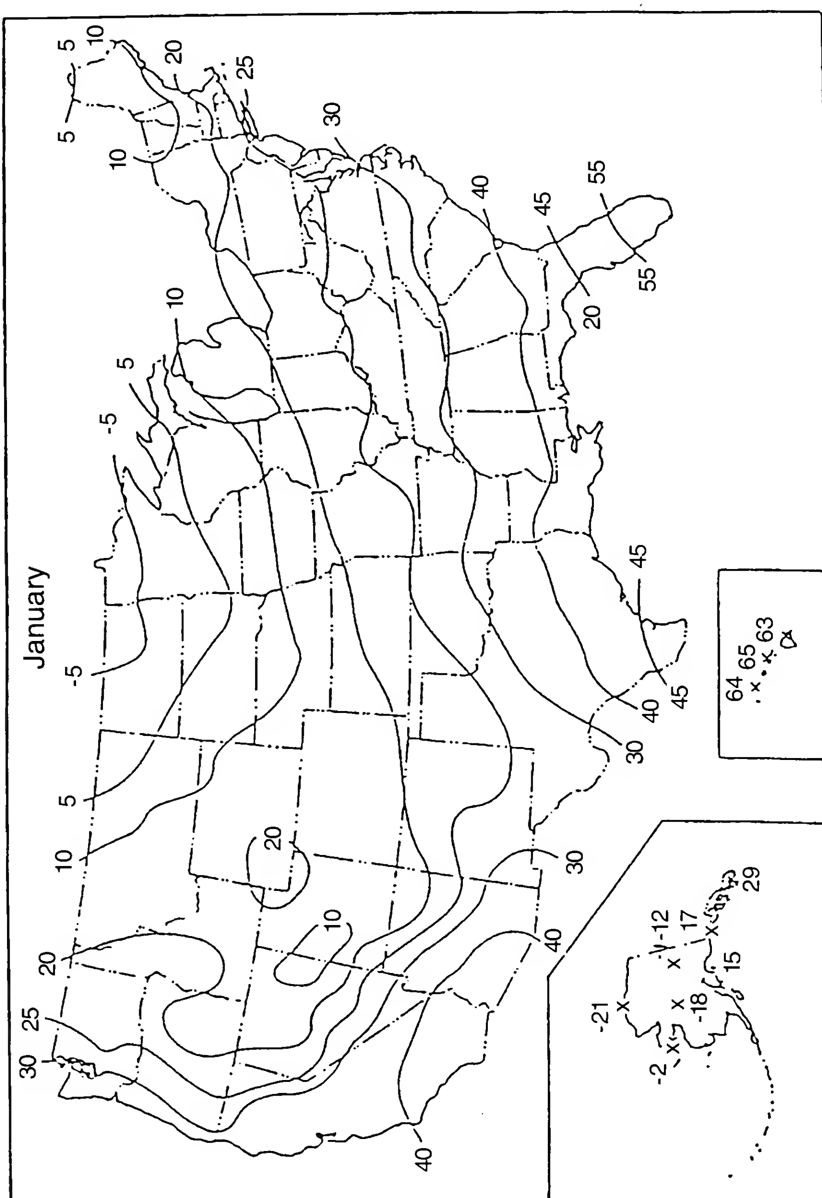


Figure 26.3.4.1A. Normal daily minimum temperatures (°F) for January.

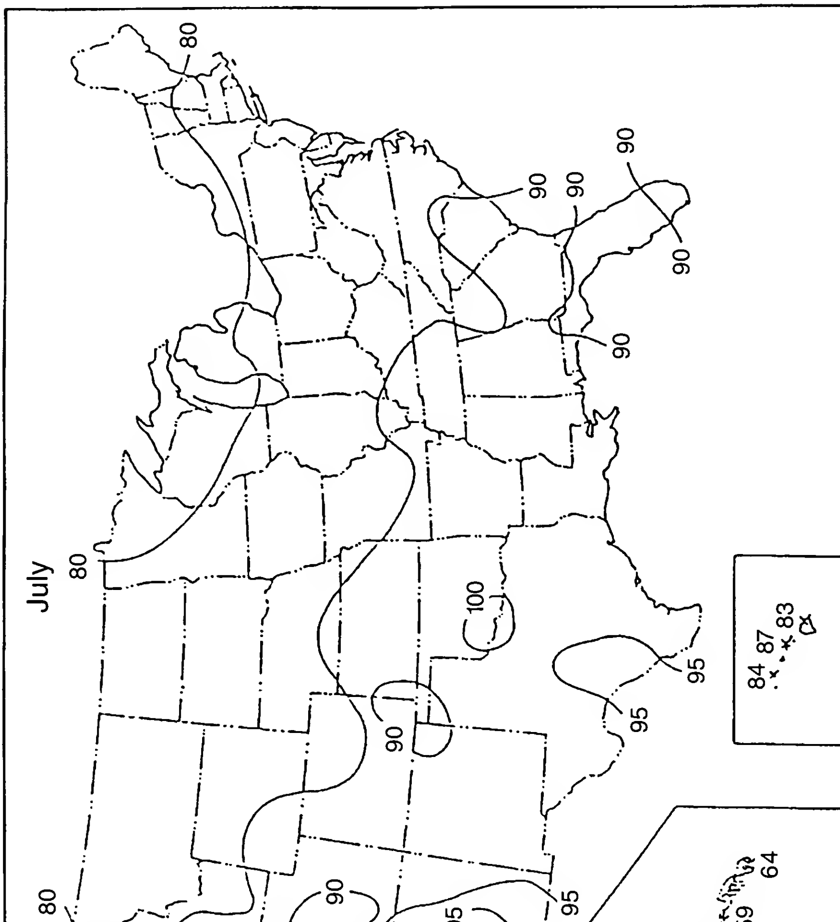


Figure 26.3.4.1B. Normal daily maximum temperatures (°F) for July.

the ACI Committee 209 Report,[16] the CEB-FIP Model Code,[18] or by a comprehensive test program. Creep strains and prestress losses which occur after closure of the structure cause a redistribution of the forces. Stresses shall be calculated for this effect based on an assumed construction schedule stated on the plans.

26.3.6 Post-Tensioning Force

(a) The structure shall be designed for both initial and final post-tensioning forces. Prestress losses shall be calculated for the construction schedule stated on the plans. The final post-tensioning forces used in service load stress calculations shall be based on the most severe condition at each location along the structure.

26.4 LOAD FACTORS

26.4.1 General

(a) In the final working condition, service or load factor load combinations shall be in accordance with Part 2 as amended below. Allowable stresses shall be in accordance with Article 26.5. When checking tensile stresses for service load, Groups II through IX, the variable load effects shall be divided by the allowable stress increases in Article 2.2.4. Strength reduction factors, ϕ, shall be in accordance with Article 26.4.3. During construction, load case combinations, allowable stresses and stability shall be in accordance with Article 26.4.4.

26.4.2 Service Load Combinations for Article 2.2.4.

26.4.2.1 Creep and Shrinkage

(a) The permanent effects of creep and shrinkage shall be added to all specified loading combinations with a load factor of 1.0.

(b) For the group loading combinations listed in Article 2.2.4, the following abbreviations shall apply:

D = DL + SDL + EL and
OF = TRF + DT + R where:
EL* = Erection Loads (final state)
TRF = Thermal – Rise or Fall
DT = Thermal – Differential
R** = Creep Effects

A thermal differential of 0.5DT is permissible when the load combination includes full live load + impact.

*See Article 26.4.2.2
**Creep effects to be considered, in conjunction with any rib shortening, shrinkage and anticipated support settlement effects as loading designation R.

26.4.2.2 Erection Loads at End of Construction

(a) The final state erection loads are defined as the final accumulated "built-in" forces and moments resulting from the construction process.

26.4.2.3 Additional Thermal Loading Combination

(a) In addition to Group Loads IV, V, and VI at service load, the following combination and stress shall apply:

(DL + SDL + EL) + E + B + SF + R + S + (DT) @ 100% Allowable Stress

(Letters in are as per Article 26.1.2, others are as per Article 2.2.3)

26.4.3 Strength Reduction Factors

(a) The basic strength reduction factors, ϕ_f and ϕ_v for flexure and shear, respectively, shall consider both the type of joint between segments and the degree of bonding of the post-tensioning system provided. The appropriate value of ϕ_v listed below shall be used for torsional effect calculations in Section 26.8.

(b) Since the post-tensioning provided may be a mixture of fully bonded tendons and unbonded or partially bonded tendons, the strength reduction factor at any section shall be based upon the bonding conditions for the tendons providing the majority of the prestressing force at the section. All internal tendons shall be designed and constructed as bonded tendons.

(c) In order for a tendon to be considered as fully bonded to the cross-section at a section, it must be bonded beyond the critical section for a development length. The development length shall be calculated by a rational approach based upon tendon pull out tests.

(d) Cast-in-place concrete joints and wet concrete joints shall be considered as Type A joints.

(e) Epoxy joints between precast units shall be considered as Type B joints.

(f) Dry joints between precast units shall be considered as Type B joints.

(g) Strength reduction factor, ϕ, shall be taken as follows:

Type	ϕ_f Flexure	ϕ_v Shear
Fully Bonded Tendons		
Type A Joints	0.95	0.85
Type B Joints	0.90	0.80
Unbonded or Partially Bonded External Tendons		
Type A Joints	0.90	0.90
Type B Joints	0.85	0.75

The appropriate value of ϕ_v from the above table shall be used for torsional effect calculations in Section 26.8.

(h) The strength reduction factor for bearing, ϕ_b shall be taken as 0.70 for all types of construction. This value shall not be applied to bearing stresses under anchorage plates for post-tensioning tendons.

26.4.4 Construction Load Combinations, Stresses and Stability

26.4.4.1 Erection Loads During Construction

(a) Erection Loads as defined by AREA and as stated on the plans shall be as follows:

1. Dead load of structure (DL): Unit weight of concrete (including rebar) 155 pcf or as determined for the project. Weight of diaphragms, anchor blocks, or any other deviations from the typical cross-section shall be included in the dead load calculations.

2. Differential load from one cantilever (DIFF): This only applies to balanced cantilever construction. The load is 2% of the dead load applied to one cantilever.

3. Superimposed dead load (SDL): This does not normally apply during construction. If it does, it should be considered as part of the dead load (DL).

4. Distributed construction live load (CLL): This is an allowance for miscellaneous items of plant, machinery and other equipment apart from the major specialized erection equip-

ment. The following magnitudes shall be used as minimum unless loads of different magnitudes can be verified. Distributed load allowance 10 psf. In cantilever construction, distributed load shall be taken as 10 psf on one cantilever and 5 psf on the other. For bridges built by incremental launching, construction live load may be taken as zero.

5. Specialized construction equipment (CE): This is the load from any special equipment such as a launching gantry, beam and winch, truss or similar major item. This also includes segment delivery trucks and the maximum loads applied to the structure by the equipment during the lifting of segments.
6. Impact Load from equipment (IE): To be determined according to the type of machinery anticipated. For very gradual lifting of segments, where the load involves small dynamic effects, the impact load may be taken as 10 percent.
7. Longitudinal construction equipment load (CLE): The longitudinal force from the construction equipment.
8. Segment unbalance (U): This applies primarily to balanced cantilever construction but can be extended to include any "unusual" lifting sequence which may not be a primary feature of the generic construction system. The load "U" is the effect of any out of balance segments or other unusual condition as applicable.
9. Wind uplift on cantilever (WUP): 5 psf minimum (balanced cantilever construction applied to one side only).
10. Accidental release or application of a precast segment load or other sudden impact from an otherwise static segment load of WTd1: Force plus Impact = 2WT1.
11. Creep (R): In accordance with Article 26.3.5. Creep effects shall be considered as part of rib shortening (R).
12. Shrinkage (S): In accordance with Article 26.3.5
13. Thermal (T): The sum of the effects due to thermal rise and fall (TRF) and differential temperature (DT) from Article 26.3.4.

26.4.4.2 Construction Load Combinations and Allowable Stresses

(a) Stresses shall be checked under the service load combinations given in Table 26.4.4.2. The distribution and application of the individual erection loads (Article 26.4.4.1) appropriate to a construction phase shall be such as to produce the most unfavorable effects. Table 26.4.4.2 is a guide; if more unfavorable conditions may occur with the particular construction system, these shall be taken into account. The maximum allowable construction load compressive stress shall be 0-5 f'_c.

(b) Load factor design need not be used for construction conditions with the exception of Article 26.4.4.3.

26.4.4.3 Construction Load Combinations Load Factor Design Check

(a) Using strength reduction factors (ϕ) in accordance with Article 26.4.3, the strength provided shall not be less than required by the following load factor combinations:

For maximum forces and moments:

$$1.1\ (DL + DIFF) + 1.3CE + 2A$$

For minimum forces and moments:

$$DL + CE + 2A$$

Table 26.4.4.2. Allowable Tensile Stresses for Construction Load Combinations

Combination	Dead Loads DL	Dead Loads DIFF	Dead Loads U	Live Loads CLL	Live Loads CE	Live Loads IE	Live Loads CLE	Wind Loads W	Wind Loads WUP	Wind Loads WE	(1)**** Allowable Stress	(R+S+T)	(2)**** Superstructure Only Including (R+S+T) Allowable Stress	B_t***	B	SF	Segmental Substructure Only (3)****Excluding (R+S+T) Allowable Stress	Segmental Substructure Only (4)****Including (R+S+T) Allowable Stress	Comments
a	1	1	0	1	1	1	0	0	0	0	$6\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	B_E	1	1	$6\sqrt{f'_c}$	$7\sqrt{f'_c}$	
b	1	0	1	1	1	1	0	0	0	0	$6\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	B_E	1	1	$6\sqrt{f'_c}$	$7\sqrt{f'_c}$	
c	1	1	0	0	0	0	0	0.7*	0.7	0	$6\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	1	1	1	$7\sqrt{f'_c}$	$7\sqrt{f'_c}$	
d	1	1	0	1	1	0	0	0.7*	1	0.7	$7\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	B_E	1	1	$7\sqrt{f'_c}$	$7\sqrt{f'_c}$	Equipment not working
e	1	0	1	1	1	1	0	0.3**	0	0.3	$7\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	B_E	1	1	$7\sqrt{f'_c}$	$7\sqrt{f'_c}$	Normal Erection
f	1	0	0	1	1	1	1	0.3**	0	0.3	$7\sqrt{f'_c}$	1	$7\sqrt{f'_c}$	B_E	1	1	$7\sqrt{f'_c}$	$7\sqrt{f'_c}$	Moving Equipment

The allowable stresses in Columns 1 and 2 apply to the summation of all the loads multiplied by their tabulated coefficients in all the columns to the left. Similarly for Columns 3 and 4 with the exceptions of (R+S+T) as noted.

*Reduction is to allow for lesser probability of maximum wind during construction period.

**Reduction is to allow for limiting wind beyond which construction is halted.

***The B_E term is as defined in AASHTO Section 3.22.

****When less than 50% of the tendon capacity is provided by internal tendons, the maximum allowable construction stresses shall be $3\sqrt{f'_c}$ for Type A joints, and 0 for Type B joints.

26.5 ALLOWABLE STRESSES

26.5.1 Prestressing Steel

(a) The allowable stresses for prestressing steel shall be in accordance with the provisions of Part 17.

26.5.2 Prestressed Concrete

26.5.2.1 Temporary Stresses Before Losses Due to Creep and Shrinkage, at the Time of Application of the Prestress

(a) Maximum Compression: 0 55 f'_{ci}.

(b) Longitudinal stresses in the PRECOMPRESSED tensile zone:

1. Type A joints with minimum bonded mild steel auxiliary reinforcement through the joints sufficient to carry the calculated tensile force at a stress of 0.5 f_y; internal tendons.

$$3\sqrt{f'_{ci}} \text{ maximum tension}$$

2. Type A joints without the minimum bonded mild steel auxiliary reinforcement through the joints; internal or external tendons: 0 tension
3. Type B joints, external tendons not less than: 200 psi minimum compression
4. Tension in other areas without bonded nonprestressed reinforcement: 0 tension.
5. Where the calculated tensile stress exceeds the allowable tensile value, bonded reinforcement shall be provided at a stress of 0.5 f_y to resist the total tensile force in concrete computed on the assumption of an uncracked section. In such cases, the maximum tensile stress shall not exceed $6\sqrt{f'_{ci}}$.

26.5.2.2 Stresses at the Service Level After Losses

(a) Maximum Compression:

$$0.4f'_c$$

(b) Longitudinal stresses in the PRECOMPRESSED tensile zone:

1. Type A joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated tensile force at a stress of 0.5 f_y; internal tendons:

$$3\sqrt{f'_c} \text{ maximum tension}$$

2. Type A joints without minimum bonded auxiliary reinforcement through joints: 0 tension
3. Type B joints, external tendons, not less than: 200 psi minimum compression
4. Tension in other areas without bonded reinforcement: zero tension
5. Where the calculated tensile stress exceeds this value, bonded reinforcement shall be provided at a stress of 0.5 f_y to resist the total tensile force in the concrete computed on the assumption of an uncracked section. In such cases, the maximum tensile stress shall not exceed $6\sqrt{f'_c}$.

(c) Transverse tension in the precompressed tensile zone:

$$3\sqrt{f'_c} \text{ maximum tension}$$

26.5.2.3 Anchorage

(a) The bearing stresses under the anchor plates shall be in accordance with the provisions of Article 17.5.10 as modified by this section. The stresses calculated at application of the post-tensioning force and at the service load shall be limited to 5000 psi. and 6250 psi., respectively.

(b) Anchorage devices which function on the basis of confinement reinforcing need not conform to the bearing stress limitations for plate type anchorage devices specified in (a). Acceptance of such anchorage devices shall be based on review of test data or on the basis of documented performance on major bridge projects.

(c) The concrete splitting force shall be calculated in accordance with Article 26.10.2; by test results based on similar anchorages, tendon trajectory, and concrete section geometry; or by more rigorous analytical procedures.

(d) Reinforcement shall be provided to resist the anchorage splitting forces.

(e) Tensile stress in anchorage splitting reinforcement at the time of application of the prestress: 0.6 f_y, where f_y shall not exceed 60,000 psi.

26.6 PRESTRESS LOSSES

(a) Prestress losses shall be computed in accordance with the provisions of Part 17. Lump sum losses shall only be used for preliminary design purposes. Losses due to creep, shrinkage, and elastic shortening of the concrete as well as friction, wobble, anchor set and relaxation in the tendon shall be calculated for the construction method and schedule shown on the plans in accordance with time-related procedures for calculation of prestress losses.

26.7 FLEXURAL STRENGTH

26.7.1 General

(a) Flexural strength of segmental concrete bridges shall be calculated in conjunction with Part 17. The flexural capacity required by the load factor provisions of Article 26.4.1 shall be less than or equal to ϕ_f times the flexural capacity provisions of Part 17. The values of ϕ_f shall be taken from Article 26.4.3.

26.7.2 Strain Compatibility

(a) As an alternative to use of Part 17, flexural strength of bonded tendon bridges may be calculated in accordance with the strain compatibility provisions of Section 10.2 of the ACI 318 Building Code.[26] Strain compatibility analysis may also be used for computation of bridges with unbonded tendons provided that the analysis correctly recognizes the differences in strain between the tendons and the concrete section, and provided that the analysis recognizes the effect of tendon anchorage lateral restraints and deflection geometry changes on the effective stress in the tendons.

26.7.3 Center of Gravity Correction for Strand Tendons

(a) Draped strand tendons shall be assumed to be at the bottom of the duct in negative moment areas, and at the top of the duct in positive moment areas. For both strength and allowable stress calculations, the location of the tendon center of gravity with respect to the center line of the duct shall be assumed as illustrated by Figure 26.7.3 (negative moment area shown).

26.8 SHEAR AND TORSION

26.8.1 Scope

(a) The provisions of Section 26.8 shall apply to the design of prestressed concrete segmental bridges subjected to shear or combined shear and torsion. Design for shear of combined shear and torsion shall be based on ultimate load conditions. The provisions of Article 26.8.2 shall apply to all parts of Section 26.8.

(b) Regions with one-way beam or thin plate type action or similar conditions in which the plane sections assumption of flexural theory can be applied shall be designed for shear or shear and torsion according to Article 26.8.1, and either the traditional approach of Article 26.8.3 or the strut-

Duct Size	Z
3" OD and Less	1/2
Over 3" OD to 4"	3/4
Over 4" OD	1"

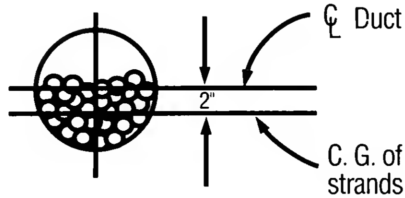


Figure 26.7.3. Negative moment region.

and-tie model approach of Article 26.8.4. Detailing of all shear and torsion reinforcement must meet the requirements of Article 26.8.2.

(c) Discontinuity regions where the plane sections assumption of flexural theory is not applicable such as regions adjacent to abrupt changes in cross sections, openings, dapped ends, regions where large concentrated loads, reactions, or post-tensioning forces are applied or deviated, diaphragms, deep beams, corbels or joints shall be designed for the applied forces causing shear or shear and torsion according to Article 26.8.2 and the strut-and-tie model approach of Article 26.8.4. In addition, special discontinuity regions like deep beams, brackets and corbels should be designed for the applicable parts of Article 26.8.5.

(d) Interfaces between elements such as webs and flanges, between dissimilar materials, between concretes cast at different times, or at an existing or potential major crack shall be designed for shear transfer in accordance with Article 26.8.6.

(e) Slab type regions subjected to local concentrated forces such as concentrated loads or column reactions shall be designed for two-way punching shear in accordance with Article 26.8.7.

(f) The applied shear on a cross section shall consist of the shear due to factored ultimate dead load (V_{uDL}) including continuity effects, factored ultimate live load (V_{uLL}) and any other factored ultimate load cases specified. Torsional moments (T_u) shall be included in design for factored ultimate load when their magnitude exceeds the value specified in Paragraph 26.8.2(j).

(g) The applied shear due to the component of the effective longitudinal prestress force which acts in the direction of the section being examined (V_p) shall be considered as a load effect.

(h) The vertical component of inclined tendons shall only be considered to reduce the applied shear on the webs for tendons which cross the webs and are anchored or fully developed by anchorages, deviators, or internal ducts located in the outer 1/3 of the webs.

26.8.2 General Requirements

(a) For members subjected to combined shear and torsion, the resulting shear forces in the different elements of the structure from the combined shear flows from shear and from torsion shall be considered. The individual elements shall be designed for the resultant shear forces.

(b) The effects of axial tension due to creep, shrinkage and thermal effects in restrained members shall be considered wherever applicable.

(c) The component of the effective prestressing force in the direction of the shear force shall be considered in accordance with Paragraph 26.8.1(f).

(d) The components of inclined flexural compression or tension in variable depth members shall be considered.

(e) The effects of any openings or ducts in members shall be considered. In determining the effective web width, b_w or b_e the diameters of ungrouted ducts or one-half the diameters of grouted ducts shall be subtracted from the web width at the level of these ducts.

(f) The values of $\sqrt{f'_c}$ used in any part of Section 26.8 shall not exceed 100 psi.

(g) The design yield strength of nonprestressed transverse shear or torsion reinforcement shall not exceed 60 ksi. The shear and torsion resistance contribution of prestressed transverse shear or torsion reinforcement shall be based on substitution of the effective stress after allowance for all prestress losses plus 60 ksi, but not to exceed f^*_y, in place of f_y in transverse reinforcement expressions.

(h) In pretensioned elements, the reduced prestress in the transfer length of the prestressing tendons shall be considered when computing f_{pc} and V_p. The prestress force due to a given tendon shall be assumed to vary linearly from zero at the point at which bonding commences to a maximum at a transfer length which may be assumed as 50 diameters for 1/2 inch diameter strand.

(i) Shear effects may be neglected in areas of members where the factored shear force V_u is less than $\phi V_c/2$ (V_c is defined in Paragraph 26.8.2[1]). Nominal minimum stirrup capacity of not less than the equivalent of two No. 4 Grade 60 bars at 1 ft. on centers shall be provided per web in such areas or the minimum shrinkage and temperature reinforcement required by Section 2.12.

(j) Torsional effects may be neglected in members where the factored torsional moment T_u is less than $\phi T_c/3$. In lieu of a more detailed calculation, T_c may be taken as

$$T_c = 2K\sqrt{f'_c}\,(2A_o b_e)$$

K shall be computed as

$$\sqrt{1 + (f_{pc}/2\sqrt{f'_c})} \text{ but } K \leq 2.0.$$

However, K shall not exceed 1.0 at any section where the stress in the extreme tension fiber due to factored load and effective prestress force exceeds $6\sqrt{f'_c}$) in tension. The influence of axial tension, N_{ut}, shall be accounted for by replacing f_{pc} by ($f_{pc} - N_{ut}/A_g$). The influence of axial compression, N_{uc}, shall be accounted for by replacing f_{pc} by the term ($f_{pc} + N_{uc}/A_g$). A_o is the area enclosed by the shear flow path defined by the centroids of the longitudinal chords of the space truss model resisting the applied torsion. In lieu of a more precise analysis, A_o may be taken as 85% of the area enclosed by the centerline of the exterior closed transverse torsion reinforcement. b_e is the effective width of the shear flow path of the elements making up the space truss model resisting torsion. In box girders b_e may be taken as A_{cp}/p_{cp}, where A_{cp} is the area enclosed by the outside perimeter of the concrete cross section and P_{cp} is the outside perimeter of the concrete cross section. The effects of openings and ducts must be considered as required in Paragraph 26.8.2(e).

(k) In a statically indeterminate structure where significant reduction of torsional moment in a member can occur due to redistribution of internal forces upon cracking, the factored torsion moment T_u may be reduced to ϕT_c [T_c is defined in Paragraph 26.8.2(j)], provided that moments and forces in the member and in adjoining members are adjusted to account for the redistribution. In lieu of a more exact analysis, the torsional loading from a slab may be assumed as linearly distributed along the member.

(l) Transverse reinforcement shall be provided in all elements except for slabs and footings, and elements where V_u is less than 0.5 ϕV_c. In lieu of more detailed calculations, V_c may be taken as:

$$V_c = 2K\sqrt{f'_c}\,b_w d$$

K shall be computed in accordance with Paragraph 26.8.2 (j).

(m) Where transverse reinforcement is required, the minimum tensile capacity of the transverse reinforcement shall be 50 $b_w s$, where b_w and s are in inches. Greater amounts may be required to carry shear and torsion to meet the requirements of Article 26.8.3 or 26.8.5.

(n) Transverse reinforcement may consist of:

1. Stirrups perpendicular to the axis of the member or making an angle of 45° or more with the longitudinal tension reinforcement, inclined to intercept potential cracks.
2. Welded wire fabric sheets or cages with wires located perpendicular to the axis of the member.
3. Longitudinal bars bent to provide an inclined portion making an angle of 30° or more with the longitudinal tension reinforcement and inclined to intercept potential diagonal cracks.
4. Well-anchored prestressed tendons which are carefully detailed and constructed to minimize seating and time dependent losses.
5. Combinations of stirrups, tendons, and bent longitudinal bars.
6. Spirals.

(o) Transverse reinforcement shall be detailed so that the shear forces between the different elements or zones of a member are effectively transferred. Transverse shear or torsion reinforcement shall extend as a continuous tie from the extreme compression fiber (less cover) to the outermost tension reinforcement. All transverse reinforcement shall be fully anchored according to Article 2.13.1.

(p) Torsion reinforcement shall consist of longitudinal bars or tendons and:

1. closed stirrups or closed ties, perpendicular to the axis of the member;
2. a closed cage of welded wire fabric with transverse wires perpendicular to the axis of the member:
3. spirals.

(q) Transverse torsion reinforcement shall be made fully continuous and shall be anchored according to Paragraph 2.21.(b)1, where the concrete surrounding the anchorage is restrained against spalling by flange or slab or similar element. Anchorage shall be by 135° standard hooks around longitudinal reinforcement where the concrete surrounding the anchorage is unrestrained against spalling. Spacing of closed stirrups or closed ties shall not exceed one-half of the shortest dimension of the cross section, nor 12 in.

(r) At any place on the cross section where the axial tension due to torsion and bending exceeds the axial compression due to prestressing and bending, either supplementary tendons to counter the tension must be added or local longitudinal reinforcement which is continuous across the joints between segments is required.

(s) If supplementary tendons are added, they shall be distributed around the perimeter of the precompressed tension zone inside the closed stirrups. At least one tendon shall be placed near each corner of the stirrups in the precompressed tension zone.

(t) If longitudinal reinforcement is added, the bars shall be distributed around the perimeter formed by the closed stirrups. Perimeter bar spacing shall not exceed 18 in. At least one longitudinal bar shall be placed in each corner of the stirrups. The minimum diameter of the corner bars shall be 1/24 of the stirrup spacing but no less than that of a #5 bar.

(u) Maximum spacing of transverse reinforcement shall not exceed 0.5d in nonprestressed elements, 0.75h in prestressed elements nor 36 in. When V_u exceeds $6\phi\sqrt{f'_c}b_wd$, these maximum spacings shall be reduced by one-half.

(v) Flexural reinforcement, including tendons, shall be extended beyond the theoretical termination or deviation points for a distance of at least h/2. Transverse reinforcement for shear and torsion shall be provided for a distance at least h/2 beyond the point theoretically required.

(w) Shear keys in webs of precast segmental bridges shall extend for as much of the web height as is compatible with other detailing requirements. Alignment shear keys shall also be provided in top and bottom flanges.

26.8.3 Traditional Shear and Torsion Design for Plane Section Type Regions

(a) The design of beam-type members or regions for shear and torsion may be carried out according to Article 26.8.3 provided:

1. V_n does not exceed $10\sqrt{f'_c}\,b_w d$
2. $\sqrt{(V_n/b_w d)^2 + (T_n/2A_o b_e)^2}$ does not exceed $15\sqrt{f'_c}$.
3. There are no significant discontinuities such as abrupt changes in cross section or openings.
4. No concentrated load located within 2d of a support causes more than one-third of the shear at that support.
5. Where required, shear reinforcement consists of tendons and stirrups perpendicular to the axis of the member or welded wire fabric sheets or cages with wires perpendicular to the axis of the member, and conforms to Article 26.8.2.
6. Where required, torsion reinforcement consists of longitudinal bars, and closed stirrups perpendicular to the axis of the member, and conforms to Article 26.8.2.

(b) The design of cross sections subject to shear shall be based on $V_u \leq \phi V_n$ where V_u is the factored shear force and V_n is the nominal shear strength. V_u shall consider any unfavorable effects of prestressing and may consider favorable effects of prestressing in accordance with Paragraph 26.8.1(f). For the purposes of this section, V_n may be computed as:

$$V_n = V_c + V_s$$

where V_c may be determined from Paragraph 26.8.2 (l) and V_s may be determined from (d). In equations for V_c and V_s, d shall be the distance from the extreme compression fiber to the centroid of the prestressed reinforcement in the tension chord or 0.8h, whichever is greater.

(c) The applied shear V_u in regions near supports may be reduced to the value computed at a distance h/2 from the support when both of the following conditions are satisfied:

1. The support reaction, in the direction of the applied shear, introduces compression into the support region of the member, and
2. No concentrated load occurs within a distance h from the face of the support.

(d) The nominal shear contribution of the truss model with concrete diagonals at 45° inclination as determined by the shear reinforcement perpendicular to the axis of the member is

$$V_s = A_s f_{sy} d/s$$

(e) Where required by Paragraph 26.8.2 (j), torsion reinforcement shall be provided in addition to the reinforcement required to resist the factored shear, flexure and axial forces that act in combination with the torsion.

(f) The longitudinal and transverse reinforcement required for torsion shall be determined from:

$$T_u \leq \phi T_n$$

(g) The nominal torsional resistance provided by a space truss with concrete diagonals at 45° inclination and the indicated transverse reinforcement for torsion is:

$$T_n = 2A_o A_t f_{sy}/s$$

where A_o is as defined in Paragraph 26.8.2 (j).

(h) The additional longitudinal reinforcement for torsion shall not be less than:

$$A_l = (T_n p_h) / (2A_o f_{sy})$$

where p_h is the perimeter of the polygon defined by the centroids of the longitudinal chords of the space truss resisting torsion. p_h may be taken as the perimeter of the centerline of the outermost closed stirrups. A_l shall be distributed around the perimeter of the closed stirrups in accordance with Paragraph 26.8.2 (t).

(i) The area of additional longitudinal torsion reinforcement in the flexural compression zone may be reduced by an amount equal to $M_u/(0.9d\ f_{sy})$ where M_u is the factored bending moment acting at that section except that the reinforcement provided shall not be less than required by Paragraph 26.8.2 (t).

26.8.4 Strut-and-Tie Truss Model Design for Either Beam Type or Discontinuity Regions

(a) The design of any region for shear and torsion may be carried out according to Article 26.8.4 based on an analysis of the internal load paths for all forces acting on the member or region. The effects of the prestress force shall be included in accordance with Paragraph 26.8.1 (f). The internal load paths shall be idealized using appropriate strut-and-tie or space truss models consisting of:

1. Concrete and compressive reinforcement compression chords
2. Inclined concrete compressive struts
3. Longitudinal reinforcement tension chords or ties
4. Transverse reinforcement tension members or ties
5. Node regions at all joints of chords, struts and ties

(b) The proportions of the elements and the reinforcement shall be selected so that the tension ties yield before the compression chords or struts crush. Chord capacities shall be based on underreinforced sections for flexure.

(c) The size of the members and joint regions in the truss shall be chosen so that the computed forces in the struts, ties, and truss members, S_u, due to factored loads shall satisfy:

1. Compression chords

$$\phi_f(085f'_c A_{cc} + A'_s f'_s) \geq S_u$$

where ϕ_f is the appropriate ϕ value for flexure.

2. Inclined compressive struts

$$\phi_v(f_{cu} A_{cs}) \geq S_u$$

where ϕ_v is the appropriate ϕ value for shear and diagonal tension and f_{cu} is the limiting strut compressive stress from Paragraph 26.8.4 (d).

3. Reinforcement tension chords

$$\phi_f\ (A_s f_{sy} + A^*_s f^*_{su}) \geq S_u$$

where ϕ_f is the appropriate ϕ value for flexure and f^*_{su} is the average stress in prestressing steel at ultimate load considering the anchorage and bonding conditions.

4. Transverse reinforcement tension members or ties:

$$\phi_v(A_v f_{sy}) \geq S_u$$

where ϕ_v is the appropriate ϕ value for shear and diagonal tension. When such members or ties are prestressed, the effective stress after prestress losses shall be used in place of f_{sy}.

5. Node regions

$$\phi_b(f_{cn}A_{cn}) \geq S_u$$

where ϕ_b is the appropriate ϕ value for bearing and f_{cn} is the limiting compressive stress in a node region from Paragraph 26.8.4 (f).

(d) The compressive stress in an inclined compressive strut, f_{cu} shall not exceed:

1. For essentially undisturbed, uniaxial compressive stress states . . . . $0.6f'_c$
2. For compressive stress states where tensile strains in the cross-direction or transverse tensile reinforcement may cause cracking of normal crack width parallel to the strut . . . . $0.45f'_c$
3. For compressive stress states with skew cracking or skew transverse reinforcement . . . . $0.35f'_c$
4. For compressive stress states with very wide skew cracks when the strut orientation differs appreciably from the elastic orientation of the internal load path . . . . $0.25f'_c$

(e) The tension chord and all tension ties shall be effectively anchored to transfer the required tension to the truss node regions in accordance with the ordinary requirements of Part 2 for development of reinforcement (Section 2.14) and shall be detailed to satisfy the stress limits of Paragraph 26.8.4 (f).

(f) Unless special confining reinforcement is provided, the concrete compressive stress f_{cn} in the node regions shall not exceed:

1. $0.85f'_c$ in node regions bounded by compressive struts and bearing areas
2. $0.70f'_c$ in node regions anchoring only one tension tie, or
3. $0.60f'_c$ in node regions anchoring tension ties in more than one direction.

26.8.5 Special Requirement for Diaphragms, Deep Beams, Corbels and Brackets

26.8.5.1 General

(a) All discontinuity regions must be proportioned using the strut-and-tie model approach of Article 26.8.4. Special discontinuity regions like diaphragms, deep beams, corbels, brackets must also satisfy the special provision of Article 26.8.5.

26.8.5.2 Diaphragms and Deep Beams

(a) Diaphragms are ordinarily required in pier and abutment superstructure segments to distribute the high shear forces to the bearings. Vertical and transverse post-tensioning shall be analyzed using the strut-and-tie model of Article 26.8.4 and the effective prestress forces of Paragraph 26.8.1 (f). The diaphragm tendons must be effectively tied into the diaphragms with bonded nonprestressed reinforcement to resist tendon forces at the corners of openings in the diaphragms.

(b) Deep beams are members in which the distance from the point of zero shear to the face of the support is less than 2d or members in which a load causing more than one-third of the shear at a support is closer than 2d from the face of the support.

1. The strut-and-tie model of Article 26.8.4 shall be used to analyze and design deep beams.
2. The minimum tensile capacity of transverse reinforcement shall be $120b_ws$, and s shall not exceed d/4 nor 12 in.
3. Bonded longitudinal bars shall be well distributed over each face of the vertical elements in pairs. The minimum tensile capacity of this bonded reinforcement pair shall be $120b_ws$. The vertical spacing between each pair, s, shall not exceed d/3 nor 12 in.

4. In deep beam vertical elements with a width less than 10 in., the pairs of bonded bars required by (b)3 may be replaced by a single bar with the required tensile capacity.

26.8.5.3 Brackets and Corbels

(a) The strut-and-tie model of Article 26.8.4 shall be used to analyze and design brackets and corbels.

(b) The depth at the outside edge of the bearing area shall be at least half the depth at the face of the support.

(c) Corbels and brackets shall be designed to resist the calculated external tensile force N_{ut} acting on the bearing area, but N_{ut} shall not be less than 0.2 V_u unless special provisions are made to avoid tensile forces. Therefore, N_{ut} shall be regarded as a live load even when tension results from creep, shrinkage or temperature change.

(d) The steel ratio A_s/bd at the face of the support shall be at least 0.04 f'_c/f_y, where d is measured at the face of the support.

(e) Closed stirrups or ties parallel to the primary tensile tie reinforcement, A_s, with a total area not less than 0.5 A_s shall be uniformly distributed within 2/3 of the effective depth adjacent to A_s.

(f) At the front face of a bracket or corbel, the primary tension reinforcement A_s shall be effectively anchored to develop the specified yield strength, f_y, by:

1. A structural weld to a transverse bar of at least equal size, or;
2. Bending the primary bars, A_s back to form a continuous loop, or;
3. Some other positive means of anchorage.

(g) The bearing area of the load on a bracket or corbel shall not project beyond the interior portion of the primary tension bars, A_s, nor project beyond the interior face of any transverse anchor bar.

26.8.6 Shear Transfer at Interfaces

(a) Shear transfer at interfaces shall be designed in accordance with Article 2.35.4 using the ϕ values from Part 26.

26.8.7 Two-way Punching Shear

(a) Two-way punching shear slab type elements shall be designed in accordance with Article 2.35.6 using the appropriate ϕ values from this Specification.

26.9 FATIGUE STRESS LIMITS

26.9.1 Fatigue Stress Limits for Bonded Nonprestressed Reinforcement

(a) Design of bonded nonprestressed reinforcement for fatigue shall conform to the provisions of Article 2.26.2.

26.9.2 Fatigue Stress Limits for Prestressed Reinforcement

(a) Fatigue of prestressed reinforcement need not be considered for bridges designed in accordance with this Specification.

26.10 DESIGN OF LOCAL AND GENERAL ANCHORAGE ZONES, ANCHORAGE BLISTERS AND DEVIATION SADDLES

26.10.1 General

(a) Anchorage zones for post-tensioning tendons are regions of complex stresses. The post-tensioned anchorages zone may be considered as comprised of two zones.

(b) The local zone is the region immediately surrounding each anchorage device. It may be taken as a cylinder or prism with transverse dimensions approximately equal to the sum of the projected size of the bearing plate plus the manufacturer's specified minimum side or edge cover. The length of the local zone extends for the length of the anchorage device plus an additional distance in front of the anchor equal to at least the maximum lateral dimensions of the anchor. Performance of the anchorage device and furnishing of any supplementary reinforcement required in this local zone is the responsibility of the constructor and material suppliers. These responsibilities shall be set forth in the project plans and specifications.

(c) The general zone is the region in front of the anchor which extends along the tendon axis for a distance equal to the overall depth of the member. The height of the general zone is taken as the overall depth of the member. In the case of intermediate anchorages which are not at the end of a member, the general zone shall be considered to also extend along the projection of the tendon axis for about the same distance before the anchor.

(d) Design and specification of any supplementary reinforcement required in the general zone (in addition to the required local zone reinforcement) is the responsibility of the engineer of record. Proper installation of such supplementary reinforcement is the responsibility of the constructor.

(e) Reinforcement shall be provided for bursting, splitting, and spalling tensile stresses generated by tendon anchorages and deviation saddles in accordance with the following provisions of Section 26.10. The method of analysis shall consider anchorage eccentricity, tendon inclination, and tendon curvature.

(f) The proportions and supplementary reinforcement of the local zone containing the tendon anchors must be adequate to transfer the tendon force into the mass of the concrete structure. The load transfer may be achieved by either bearing plate type anchors or by special anchorage devices which in combination with special anchor reinforcement (such as spirals, stirrups or other reinforcement) transfer the local zone loads from the anchors into the general anchorage zone of the structure.

26.10.2 Forces and Reinforcement in General Anchorage Zones

(a) The general distribution of forces and the reinforcement required to provide the necessary general anchorage zone tensile capacity to counteract the bursting forces of the anchorages may be determined using the strut-and-tie model approach of Article 26.8.4.

(b) In lieu of analysis using the strut-and-tie approach, the total bursting force, F_{bst}, for an individual anchorage shall be taken as:

$$F_{bst} = 0.30\ (1 - d_a/d_{sp})Pj$$

26.10.3 Reinforcement

26.10.3.1 Local Zones

(a) The local zone shall be reinforced for the bursting forces as required for the anchor type used in accordance with the provisions of Articles 26.5.2.3 and 26.10.2. The reinforcement may consist of stirrups, ties, spirals, or combinations of these.

26.10.3.2 General Anchorage Zone Bursting and Directional Forces

(a) The structure shall be reinforced with stirrups or ties to resist general anchorage zone bursting forces and directional forces due to total post-tensioning forces anchored at a section in accordance with the provisions of Article 26.5.2.3 and 26.10.2.

26.10.3.3 Stress in Reinforcement for Bursting Forces

(a) Reinforcement for bursting forces shall be designed for maximum jacking forces at time of stressing with $f_s = 0.6\ f_{sy}$ where f_{sy} shall not exceed 60 ksi.

26.10.3.4 Post-Tensioning

(a) Post-tensioning may be provided to supplement reinforcement restraint against anchorage bursting or directional forces.

26.10.4 Reinforcement Detailing

(a) Reinforcement may be in the form of spirals, stirrups, orthogonal reinforcement, or combinations of these. Groups of anchorages shall be restrained by reinforcement stirrups or lateral post-tensioning enclosing the entire group. All orthogonal reinforcement must be mechanically anchored around reinforcement running parallel with tendons. All spirals, stirrups, or orthogonal reinforcement shall have sufficient extra length to develop full bond with the concrete, or shall be mechanically anchored by 135° bends around reinforcement. The clear distance between bars or pitch of spirals used as anchorage zone reinforcement shall be at least the maximum size of the coarse aggregate plus 1/2 in. but not less than $1^1/_2$ in.

26.10.5 Anchorages in Special Blisters

26.10.5.1 Design

(a) In addition to reinforcements provided for tensile stresses perpendicular to the tendon trajectory, blisters shall also be designed for shear and bending between the blister and web/flange interface. For these purposes, the strut-and-tie models of Section 26.8, or the rules for shear friction and special provisions brackets and corbels as set out in Part 2 shall be applied. The reinforcement required for anchorage zone tensile stress may also be used for shear friction calculations if full bond development or mechanical anchorage within the web and slab is provided for the reinforcement.

26.10.5.2 Local Bending

(a) When blisters are used, a check shall be made for the localized bending induced into the web and/or flange in the region surrounding the anchorage. Reinforcement shall be provided equivalent to the force represented by the concrete tensile stress block proportioned at a stress of not more than 0.6 f_y, where f_y shall not exceed 60 ksi.

26.10.5.3 Local Tensile Stresses Behind Anchorage Blisters

(a) Blisters should preferably be located at the juncture of the flange and the web. Calculations shall be made to assure that sufficient residual compression exists behind anchorage blisters that no localized tensile stresses occur, or sufficient reinforcement shall be provided at an allowable stress of 0.6 f_y (maximum value of f_y to be 60 ksi.) to take all the tensile force. Use of anchorage blisters projecting from one surface only, such as a flange, should preferably be restricted to anchorage of small tendons and bars. Blisters shall preferably be located sufficiently far from a joint to allow dispersal of local tensile stress effects through the reinforced slab. Minimum reinforcement shall be provided to carry twenty-five to fifty percent of the anchor load into the concrete behind the anchor. The amount of reinforcement provided shall be based on evaluation of the compressive stress level due to other tendons or loads in the local area behind the anchor, and shall increase to an amount of reinforcement sufficient to carry 50 percent of the tendon force whenever local net tensile stresses might be generated behind the anchorage.

26.10.6 Anchorages in Diaphragms

(a) Reinforcement shall be provided to ensure a full transfer of shear load from the diaphragm to the webs and flanges. The diaphragm shall be designed and reinforced for any localized bending effects due to concentrated anchorage loads. Anchorage zones in diaphragms shall be reinforced in accordance with Article 26.10.2.

26.10.7 Anchorage Bearing Reaction Force

(a) In situations where the anchorage reaction force is not parallel to the longitudinal axis of the beam, it is necessary to take into account the magnitude and direction of the anchorage bearing reaction. Reinforcement or post-tensioning shall be provided as required to contain the component of the anchorage reaction perpendicular to longitudinal axis of the girder. The reinforcement stress may be taken as 0.6 f_{sy} but not greater than 36 ksi. (for Grade 60 steel).

26.10.8 Deviation Saddles

26.10.8.1 General

(a) Deviation saddles are blisters external to the webs and flanges, normally on the inside of a box at the junction of web and flange where tendons placed external to the concrete are deviated in direction to produce the required tendon profile.

26.10.8.2 Design

(a) Reinforcement shall be provided in the form of fully anchored reinforcement and bent bars in webs or flanges to take the resultant pull out force computed at f^*_{su} from the deviated tendon(s) at a service stress of 0.5 f_{sy}. Additional reinforcement shall be provided to take any out of balance longitudinal forces by shear friction action according to the ACI 318-86 Standard Building Code, Article 11.7. Reinforcement shall also be provided to take any localized bending and axial effects transmitted from the deviation saddles to the webs and/or flanges.

26.10.8.3 Detailing

(a) All reinforcements shall have a full effective development length measured from the tendon axis or shall otherwise be fully mechanically anchored around longitudinal reinforcement located at the outside of the (box) section. Consideration shall be given to constructibility and clearances between reinforcement for adequate concrete compaction. Not more than two reinforcing bars shall be bundled and the clear distance between reinforcement shall be at least 1/2 in. greater than the maximum coarse aggregate size and in no case less than 1½ in.

26.10.8.4 Localized Effects on Transverse Design

(a) The transverse design of the section shall be checked for the transverse force imparted through deviation saddles, including any unsymmetrical effects due to sequential post-tensioning. Additional bonded reinforcement proportioned at a tensile stress of 0.6 f_{sy}, where f_{sy} shall not be taken as greater than 60 ksi, or transverse post-tensioning shall be provided equivalent to the tensile force induced in the slab.

26.11 PROVISIONAL POST-TENSIONING DUCTS AND ANCHORAGES

26.11.1 General

(a) In accordance with Article 26.11.2, the design of ducts and anchorages for bridges with internal tendons shall provide for increases in the post-tensioning force at selected locations along the bridge during construction to compensate for excessive friction and wobble losses during stressing. For bridges with either internal or external tendons, the design shall provide for future addition of external unbonded tendons in accordance with provisions of Article 26.11.3 as an allowance for addition of future dead load, or to adjust for deflection of the bridge.

26.11.2 Bridges with Internal Ducts

(a) At intervals of not more than three segments, provisional anchorage and duct capacity for negative and positive moment tendons located symmetrically about the bridge centerline shall provide for an increase in the post-tensioning force. The total provisional force potential of both positive and negative moment anchorages and ducts shall not be less than 5 percent of the total positive and negative moment forces, respectively, and shall be distributed uniformly at three segment intervals along the length of the bridge. At least one empty duct per web shall be provided with anchorages at appropriate locations. Except for non-continuous bridges, and the minimum empty duct capacity noted above, provisional positive moment duct and anchorage capacity shall not be required for 25 percent of the span length either side of pier supports. Any provisional ducts not utilized for adjustment of the post-tensioning force shall be grouted at the same time as other ducts in the span.

26.11.3 Provision for Future Dead Load or Deflection Adjustment

(a) Specific provisions shall be made for access and for anchorage attachments, pass through openings, and deviation block attachments to permit future addition of unbonded external tendons symmetrically about the bridge centerline for a post-tensioning force of not less than 5 percent of the total positive moment and negative moment post-tensioning force.

26.12 DUCT DETAILS

26.12.1 Material Thickness

26.12.1.1 Metal Ducts

(a) Metal ducts shall be galvanized corrugated semi-rigid conduit. For strand and wire tendons, the duct thickness shall be 26 gauge up to 2⅝ in. diameter. Ducts larger than 2⅝ in. diameter shall be 24 gauge. For bar tendons, the duct thickness shall not be less than 31 gauge.

26.12.1.2 Polyethylene Duct

(a) Polyethylene duct or rigid pipe used as external duct shall be high density polyethylene conforming to ASTM D 3350. Internal polyethylene duct shall have spiral corrugations. Rigid pipe may be manufactured in accordance with ASTM D 2447, ASTM F 714, or ASTM D 2239. Material thickness shall be as follows:

1. Internal polyethylene duct = 0.050 in. ± 0.010 in.
2. External polyethylene duct shall have a minimum external diameter to wall thickness ratio of 21 or less.

26.12.2 Duct Area

(a) Duct for strand and wire tendons shall be sized so that the area of the duct is at least 2½ times the area of the prestressing steel it contains.

26.12.3 Minimum Radius of Curvature

(a) Tendon ducts shall preferably be installed with a radius of curvature of 20 ft. or more. Ducts with sharper curvature down to a minimum of 10 ft. shall have confinement reinforcement detailed to tie the duct into the concrete. Duct curvature with radii less than 10 ft. may be approved by the Engineer based on review of test data. The minimum radius for corrugated polyethylene duct shall be 30 ft. The confinement reinforcement shall be proportioned to resist radial forces calculated as:

$$F_r = P/R$$

Where P is the tendon force in pounds per foot, R is the radius of curvature, in feet, and F_r is the radial force in pounds per foot. Confinement reinforcement shall be proportioned at 0.6 f_y where f_y shall not exceed 60 ksi. Spacing of confinement reinforcement shall not exceed 12 inches. Closer spacing shall be used for duct with radius of curvature less than 15 feet.

(b) When the tendon profile radius of curvature is less than 20 ft., design consideration shall also be given to lateral forces exerted by multistrand tendons on thin webs due to bunching of the strand at the top or bottom of circular ducts. Confinement reinforcement, preferably in the form of spirals, shall be provided whenever the nominal shear stress due to tendon jacking forces in the concrete cover beside the tendon exceeds $2\sqrt{f'_c}$.

26.12.4 Duct Supports

26.12.4.1 Internal Supports

(a) Internal ducts shall be rigidly supported by ties to reinforcing steel as follows:

1. Transverse slab tendons in metal duct: 2 ft.
2. Transverse slab tendons in polyethylene duct: 2 ft.
3. Longitudinal slab or web tendons in metal duct: 4 ft.
4. Longitudinal slab or web tendons in polyethylene duct: 2 ft.

26.12.4.2 External Ducts

(a) External ducts shall have a maximum unsupported length of 25 feet unless a vibration analysis is made.

26.12.5 Duct Size, Clearance and Detailing

(a) Maximum size of ducts shall not exceed 0.4 x web thickness.

(b) Where two curved tendons run parallel such that the outer one is bearing inwards toward the inner one, a minimum clearance of one duct diameter shall preferably be maintained between the ducts. If this is not possible, reinforcement shall be provided between the ducts to fully restrain the outer tendon if it has to be stressed before the inner tendon has been stressed and grouted. In cases where longitudinal tendons cross each other at least one-half duct diameter but not less than 2 in. clear space shall be provided. This restriction does not apply to transverse ducts crossing longitudinal ducts at approximately 90°.

(c) Curved tendons should not be placed around re-entrant corners or voids. If this is unavoidable, then the tendons must be provided with well anchored, full reinforcement restraint proportioned as per Article 26.12.6.1. In no case shall the distance between the re-entrant corner or void and the edge of the duct be less than 1.5 duct diameters.

26.12.6 Duct Confinement Reinforcement

26.12.6.1 Ducts in Webs of Curved Bridges

(a) When curved tendons are located in thin webs or close to internal voids reinforcement shall be provided to prevent the tendon from bursting through the concrete into the void whenever the nominal shear stress in the cover beside the tendon due to tendon jacking forces exceeds $2\sqrt{f'_c}$. The area of steel required may be estimated from:

$$A_s = P/(R \times 0.6\, f_{sy})$$

Where A_{sy} = Area of steel required, in.^{2} ft.

(b) The lateral force exerted on the concrete by the tendons may be calculated by dividing the tendon force by the radius of curvature in accordance with Article 26.12.3.

26.12.6.2 Ducts in Flanges

(a) Ducts in bottom slabs shall be located between top and bottom layers of transverse and longitudinal slab reinforcement. For ducts in the bottom flanges of variable depth segments, nominal confinement reinforcing shall be provided around the duct at each segment face. The reinforcement shall not be less than two rows of #4 hairpin bars at both sides of each duct with vertical dimensions equal to the slab thickness less top and bottom cover dimensions.

(b) When closely spaced transverse or longitudinal ducts are located in top or bottom flanges, the top and bottom nonprestressed reinforcement mats shall be tied together with vertical reinforcement consisting of #4 hairpin bars with spacing not to exceed 18 inches or 1½ times the slab thickness in each direction, whichever is the lesser.

26.13 COUPLERS

(a) Not more than 50 percent of the longitudinal post-tensioning tendons shall be coupled at one section. The spacing between adjacent coupler locations shall not be closer than the segment length or twice the segment depth. The void areas around couplers shall be deducted from the gross section area and moment of inertia when computing stresses at the time of application of the post-tensioning force.

26.14 CONNECTION OF SECONDARY BEAMS

(a) The load from secondary beams connected to the main beam (indirect support) shall be resisted by suspension stirrups or inclined bars. Not less than 2/3 of this suspension reinforcement shall be located in the immediate area of the intersection. The entire load shall be transmitted within the intersection zone specified in Figure 26.14A. Existing shear reinforcement within the intersection zone may be considered as part of the suspension reinforcement provided that the secondary beam extends for the full height of the main beam. Suspension stirrups and inclined bars shall be anchored in accordance with Section 2.21.

(b) Detailing of the connection may be accomplished by use of the strut-and-tie procedures outlined in Article 26.8.4

26.15 CONCRETE COVER AND REINFORCEMENT SPACING

26.15.1 Cover and Spacing

(a) Reinforcement cover and spacing shall conform to Section 2.6 and to Article 26.15.2.

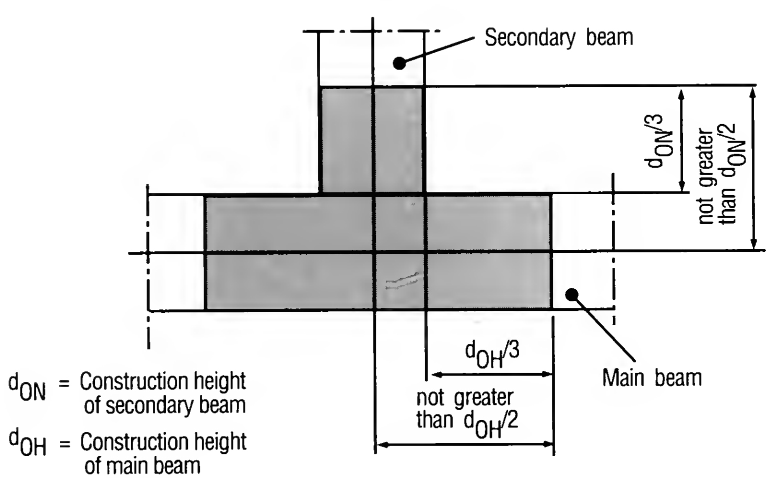


Figure 26.14A. Reinforcement details.

26.15.2 Reinforcement Details for Erection Loads

(a) The transverse analysis of the box girder shall include an evaluation of the quantity Z of equation 2-64 of Section 2.39, Part 2 for any loads applied prior to attainment of full design strength. The value of Z calculated for flanges and webs shall not exceed 130 kips per inch.

26.16 INSPECTION ACCESS

(a) Inspectability of the structure shall be assured by providing secured access hatches with minimum dimensions of 2'-6" x 4'-0". Interior diaphragms shall be provided with openings larger than the dimensions specified for access hatches. The box section shall be vented by drains or screened vents in webs at intervals not greater than 50 ft. Such venting is to prevent the build up of potential hazardous gas which might endanger inspection personnel.

26.17 BOX GIRDER CROSS SECTION DIMENSIONS AND DETAILS

26.17.1 Minimum Flange Thickness

(a) Top and bottom flange thickness shall not be less than any of the following:

1. 1/30 the clear span between webs or haunches, a lesser dimension will require transverse ribs at a spacing equal to the clear span between webs or haunches.
2. Top flange, 9 inches where transverse post-tensioning is anchored. Transverse post-tensioning or pretensioning shall be used where the clear span between webs or haunches is 15 feet or larger. Strand used for transverse pretensioning shall be 0.5 inch diameter or less.

26.17.2 Minimum Web Thickness

(a) Webs with no longitudinal or vertical post-tensioning tendons—8 inches.

(b) Webs with only longitudinal (or vertical) post-tensioning tendons—12 inches.

(c) Webs with both longitudinal and vertical post-tensioning tendons—15 inches.

26.17.3 Length of Top Flange Cantilever

(a) The cantilever length of the top flange measured from the centerline of web should preferably not exceed 0.45 the interior span of the top flange measured between the centerline of the webs.

26.17.4 Overall Cross Section Dimensions

(a) Overall dimensions of the box girder cross section should preferably not be less than required to limit live load plus impact deflection calculated using the gross section moment of inertia and the secant modulus of elasticity to 1/1000 of the span. The live loading shall be in accordance with Paragraph 2.2.3 (c), Part 2. The live loading shall be considered to be uniformly distributed to all longitudinal flexural members.

Part 26—Commentary
Recommendations for the Design of Segmental Bridges

C26.1.1 Segmental bridges contemplated under this Article include but are not limited to those erected by the following methods:

1. Balanced cantilever
2. Span-by-span with truss or falsework
3. Span-by-span lifting

4. Incremental launching
5. Progressive placement

The span length of bridges considered by these specifications ranges to approximately 800 feet. Bridges supported by stay cables are not specifically covered although many of the specification provisions are applicable to cable-stayed bridges.

Lightweight concrete has been infrequently used for segmental bridge construction. Provision for the use of lightweight aggregates represents a significant complication of both design and construction specifications. For these reasons, as well as questions concerning the economic benefit of use of lightweight aggregates for segmental bridges, their use is not explicitly covered in these specifications.

C26.1.5 Special corrosion protection is considered necessary for all bridge deck reinforcement in areas of contamination or where de-icer or other harmful chemicals may be applied. Corrosion protection should also be provided for all reinforcement of bridges located in coastal areas or over sea water, or in heavily industrialized areas.

See the ACI Committee 222 report "Corrosion of Metals in Concrete"[1] for a comprehensive discussion of methods of corrosion protection.

C26.2.1 Results of elastic analyses should be evaluated with consideration of possible variations in the modulus of elasticity of the concrete, and variations on the concrete creep and shrinkage properties, as well as the impact of variations in the construction schedule on these (and other) design parameters.

C26.2.2 Strut-and-tie models provide one means of analyzing areas near concentrated loads, bearing areas, diaphragms, corners, bends, openings, anchorage zones for post-tensioning tendons, and other areas where non-linear strains exist, as well as the cracked global structural system. Morsch proposed an extension of this concept in 1989.[2,3,4]

C26.2.3.1 The procedures of Article 3-10.2 of the 1983 Ontario Highway Bridge Design Code provides an equation for determining the effective flange width for use in calculating bending resistances and bending stresses.

C26.2.3.2 Note that b as used in this Article is the flange width on either side of the web. (b_1, b_2, or b_3 in Figure 26.2.3.2C).

The pattern of stress distribution in Figure 26.2.3.2D is intended only for calculation of stresses due to anchorage of post-tensioning tendons, and may be disregarded in the general analysis to determine design moments, shears and deflections.

Superposition of local slab bending stresses due to track loads (two-way slab action) and the primary longitudinal bending stresses is not normally required.

C26.2.4 See references[8,9] for background on transverse analysis of concrete box girder bridges.

C26.2.5.1 Analysis of concrete segmental bridges requires consideration of variation of design parameters with time, as well as a specific construction schedule and method of erection. This, in turn, requires the use of a computer program developed to trace the time-dependent response of segmentally erected prestressed concrete bridges through construction, and under service loads. Among the many programs developed for this purpose, several are in the public domain, and may be purchased for a nominal amount.[13,14,15]

A comprehensive series of equations for evaluating the time-related effects of creep and shrinkage is presented in the ACI Committee 209 report, "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structures."[16] A procedure based on graphical values for creep and shrinkage parameters is presented in the CEB-FIP Model Code.[17]

Recent research results[18] have suggested that the ACI 209 predictions underestimate the creep and shrinkage strains for the large scale specimens used in segmental bridges. The ACI 209 creep predictions were consistently about 65 percent of the experimental results in these tests. The report suggests modifications of the ACI 209 equations based on the size or thickness of the members.

C26.3.2 The use of lightweight concrete is not covered in these specifications for the reasons outlined in the commentary to Article 26.1.1

The value of 155 pcf for the unit weight of concrete is intended to provide for more heavily reinforced sections than would be anticipated in more conventional concrete superstructures.

C26.3.3 Erection loads may be imposed on opposing cantilever ends by use of the Formtraveler, diagonal alignment bars, a jacking tower, or by external weights. Cooling of one cantilever with water has also been used to provide adjustment of misalignment. Any misalignment of interior cantilevers should be corrected at both ends before constructing either closure. The frame connecting cantilever ends at closure pours should be detailed to prevent differential vertical movement between cantilevers due to forces including thermal gradient until the final structural connection is complete. The magnitude of closure forces should not induce stresses in the structure in excess of those stipulated in these specifications.

C26.3.4.3 For major bridges, tests or use of previous test data to determine more precise thermal coefficients is recommended.

C26.3.4.4 Additional field research is recommended to verify the temperature gradients specified in the referenced NCHRP report for four temperature zones in the United States. Railroad bridges differ from highway bridges when the deck is ballasted and require special attention. While the need for consideration of thermal gradients in design of concrete box girder bridges has been clearly demonstrated, opinion is divided as to the need for use of complex gradients and relatively high temperature differentials outlined in NCHRP Report 276. However, the use of the provisions of Appendix A of NCHRP Report 276 is conservative and is recommended for unballasted decks until such time as additional research data on thermal gradients and temperature differentials becomes available.

Transverse analysis for the effects of differential temperature outside and inside box girder Articles is not considered generally necessary. However, such an analysis may be necessary for relatively shallow bridges with thick webs.[9,19,20,21] In that case, a plus or minus 10 degree Fahrenheit temperature differential is recommended. Additional field research is recommended to determine temperature differentials between the inside and outside surfaces of segmental concrete box girder Articles in U. S. temperature zones.

C26.3.5 A variety of computer programs and analytical procedures have been published to evaluate creep and shrinkage effects in segmental concrete bridges.[13,14,15,16,17,22]

For permanent loads, the behavior of segmental bridges after closure may be approximated by use of an effective modulus of elasticity, E_{eff}, which may be calculated as:

$$E_{eff} = E_{cm}/\phi_c$$

Where ϕ_c is the creep coefficient, and E_{cm} is the 28 day secant modulus of elasticity of the concrete calculated from:

$$E_{cm} = 57000\sqrt{f'_c}$$

where E_{cm}, E_{eff} and f'_c are all in psi.

C26.3.6 Prestress losses vary significantly with different values of the creep coefficient, type of prestressing steel (low relaxation steel is recommended), and with the creep model (ACI 209 or CEB-FIP). Further, the prestress losses vary significantly at different sections along the superstructure.

C26.4.3 The values of ϕ_f and ϕ_v presented in Article 26.4.3 are based on consideration of relatively limited test results[24,25,26] and are considered interim provisions until further comprehensive tests, analyses, and experience with completed structures are obtained.

The proposed ϕ_f values for flexure for segmental bridges with fully bonded tendons with cast-in-place concrete joints, wet concrete joints or epoxy joints are based on the current AASHTO value of 0.95 for monolithic post-tensioned construction. *This specification assumes the practice of requiring epoxy for all joints having internal tendons passing through them is valid.* Comprehensive tests[27] of a large continuous three span model of a twin cell box girder bridge built from precast segments with fully bonded internal tendons and epoxy joints indicated that cracking was well distributed throughout the segment lengths, no epoxy joint opened at failure, and the load-deflection curve was identical to that calculated for a monolithic specimen. The complete ultimate strength of the tendons was developed at failure. The model had substantial ductility and full development of calculated deflection at failure. Recent tests[23,24] on single span segmental girders with varied tendon arrangements (internal, mixed and external tendons) and with dry joints indicate that the deflection at failure was less than would be expected for monolithic girders. Flexural cracking concentrated at joints, and final failure came with a central joint opening widely and crushing occurring at the top of the joint. The somewhat limited ductility is reflected in the reduced ϕ factors for Type B (dry) joints as well as reduced ϕ factors with unbonded tendons which allow the concentration of articulation at a single joint opening. The reduction in nominal strength for unbonded construction is adequately reflected in the determination of unbonded tendon stress at ultimate using AREA calculation procedures.

The proposed ϕ_v values for shear utilize the current AREA value of 0.85 for monolithic construction as the accepted value for Type A joints (cast-in-place, wet concrete or epoxy joints) in bonded tendon construction based on the very favorable experience in the ultimate shear tests reported in reference 27. Comparative shear tests of epoxy and dry joints indicate the epoxied joints develop the full strength of monolithically cast specimens. However, dry joints developed less strength and allowed appreciable slip along the joint. Because of this, lower ϕ_v factors are specified for dry joints (Type B).

The development length computation for defining a bonded tendon assumes that the duct is completely filled with grout and the grout completely surrounds all the strands. Therefore, the development length of a tendon is similar to that of an individual strand.

C26.4.4.1 The differential load between cantilevers is to allow for possible variations in cross-section weight.

C26.5.2.3(b) The bell anchor for threadbar tendons is an example of a confinement anchorage device that has demonstrated satisfactory performance over many years on major bridge projects. Other confinement anchorages which have demonstrated satisfactory performance utilize spiral reinforcement in conjunction with plate or casting type anchorages which do not comply with the bearing stress limitations of Paragraph 26.5.2.3 (a).

C26.5.2.3(c) NCHRP Project 10-29, "Anchorage Zone Reinforcement for Post-Tensioned Concrete Girders" is now underway at the University of Texas at Austin to develop more comprehensive recommendations for proportioning reinforcement for anchorage splitting stresses. Previous work at the University of Texas at Austin[27,28,29] includes recommendations for design of anchorage zone reinforcement that may be utilized until NCHRP Project 10-29 is completed.

Bursting or splitting forces occur in front of individual anchors inside the local zone. The magnitude of these forces depends on the shape and design of the particular anchor. For plate type anchors these bursting forces and the required reinforcement can be determined by computation or by test. For confinement anchors, bursting forces in the local zone are normally not accessible by computations. Their adequacy can only be determined by representative tests. It is the suppliers responsibility to determine the required bursting reinforcement in the local zone for such special anchors.

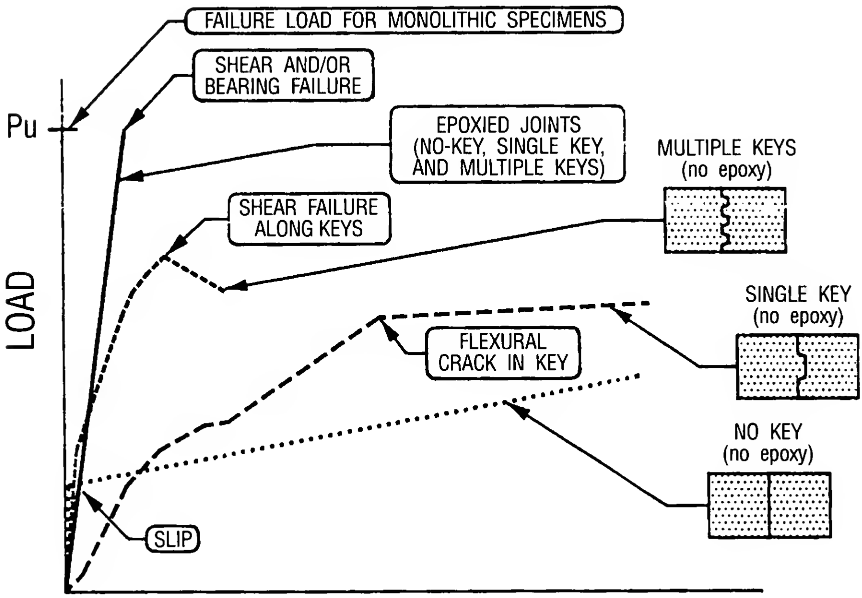


Figure C26.4.3[26] Shear key behavior.

General zone bursting forces exist beyond the individual tendon local zones. The general zone bursting forces are dependent primarily on the overall concrete dimensions and the magnitude, direction and location (eccentricity) of total prestressing force anchored and not on the particular anchor design. The reinforcement for these general zone bursting forces is part of the overall structural design, and is the responsibility of the Engineer. For design purposes, it may be conservatively assumed that any local zone reinforcement provided does not contribute to the strength of the general zone.

C26.7.1 The minimum reinforcement provisions of Part 17 were developed to avoid a brittle failure in a grossly under-reinforced simple span precast, prestressed section. Application to segmental concrete bridges results in requirements of more bonded reinforcement for bridges with more conservative (arbitrary) design tensile stress levels which is contrary to load requirements. Minimum reinforcement requirements are adequately covered by the allowable stresses and load factor requirements of these specifications.

Additional research is recommended to verify the value of f^*_{su} unbonded members. The German DIN Specification allows a stress increase of only 6 ksi for unbonded cantilever tendons, and no stress increase for fully continuous unbonded tendons.

C26.8.1 All design for shear and torsion of prestressed concrete segmental bridges is based on ultimate load conditions because little information is available concerning actual shear stress distributions at working or service load levels.

C26.8.1(b) Regions with beam-type action are basically those where the Bernoulli hypotheses that linear strain profiles exist are valid. See B-regions in Figures C26.8.1A, C26.8.1B, and C26.8.1C.

C26.8.1(c) Discontinuity regions, where the assumption that strain profiles are linear is invalid, usually exist for about a distance h from a concentrated load or point of geometrical discontinuity. See D-regions in Figures C26.8.1A, C26.8.1B, and C26.8.1C. Moving wheel loads need not be considered as large concentrated loads. The use of strut-and-tie models in design is well described in "Towards a Consistent Design of Structural Concrete," by J. Schlaich, K. Schafer, and M. Jennewein, Vol. 32, No. 3 *PCI Journal*, May/June 1987, pp. 74–150.[2] Note that a structure can be made up of both beam-type and discontinuity regions. The strut-and-tie model procedures must be used in the discontinuity regions. Either the traditional beam approach of the strut-and-tie approach can be used in the beam-type regions.

C26.8.1(d) In addition for obvious checks for shear transfer when dissimilar materials are utilized, adequate shear transfer reinforcement must be provided perpendicular to the vertical planes of web/slab interfaces to transfer flange longitudinal forces at ultimate conditions. This shear transfer shall account for the shear force, F, as shown in Figure C26.8.1D, as well as any localized shear effects due to prestress anchorages at that Article.

Article 11.7 of ACI 318 is generally termed the "shear-friction" method but does provide in Article 11.7.3 that a wide range of shear transfer design methods may be utilized. In some cases, the designer may find the strut-and-tie method of Article 26.8.4 useful in proportioning transverse reinforcement to assist in transfer of horizontal shear between elements.

C26.8.1(f) The shear effect of moving vehicle loads may be considered by development of maximum factored shear envelopes and the use of these values in determining the factored ultimate live load shear on the section.

Prestressing is considered as an applied load with a carefully controlled magnitude and direction. The components of the prestress force can add to or subtract from the shear on a cross section. In cantilevered segmental construction, the prestress vertical component can reverse the applied shear direction near the supports.

C26.8.2(f) The limitation on the effective diagonal tension and aggregate interlock components of shear strength contributed by the concrete has been adopted by ACI Committee 318.

C26.8.2(h) Research is recommended on the transfer length of 0.6 inch diameter strand.

C26.8.2(i) A simplified determination of V_c is presented which eliminates the need to check V_{ci} and V_{cw} as in the present AREA Specifications and which eliminates the complex V_{ud}/M_u term. This expression has been checked against a wide range of test data and has been found to be a conservative yet simpler expression.

C26.8.2(v) In place of requiring additional longitudinal reinforcement for shear as indicated by the mechanics of the truss model, the requirement of extending all flexural reinforcement beyond the theoretical bend or cut off points for a distance of h/2 automatically satisfies this need. Since actual shear and torsion may vary from the assumed calculation, it is also recommended that transverse reinforcement be provided for the same distance beyond the zone theoretically required.

C26.8.3 This Article is a simplified version of the present AREA approach for section design in beam-type regions. It is based on the simplified V_c term introduced in Paragraph 26.8.2(l). Provision of a "traditional" but less complex approach for beam-type regions is desirable since designers may find its application easier than strut-and-tie models for moving loads.

C26.8.3(h) In determining the required amount of longitudinal reinforcement, the beneficial effect of longitudinal prestressing may be taken into account by considering it equivalent to an area of reinforcing steel with a yield force equal to the effective prestressing force.

C26.8.4 This Article combines the recommendations of Schlaich, Schafer, and Jennewein with recommendations of Marti[3] as developed by ACI Committee 318, Subcommittee E for a future edition of the ACI Building Code. The proposed stress limits on struts and nodes may be subject to further refinement.

C26.8.4(a) Figures C26.8.4A[2] and C26.8.4B[2] illustrate the analysis using strut-and-tie models. Figure C26.8.4.C[2] shows a compression strut in a web with a tension tie in the stirrups. Figure C26.8.4D[2] gives examples of basic types of nodes. An inclination angle ϕ (Figure C26.8.4A) of 30 to 35 degrees is recommended for the inclined compressive struts in prestressed members.

C26.8.5.2(a) Figure C26.8.5.2A[2] illustrates application of the strut-and-tie model to analysis of forces in the diaphragm of a box girder bridge.

C26.8.5.2(b)1 Figure C26.8.5.2B[2] shows application of the strut-and-tie model to analysis of deep beams.

C26.8.5.3(a) Figure C26.8.5.3[2] illustrates application of strut-and-tie models to analysis of corbels.

C26.9.1 Calculation of fatigue stress limits in bonded reinforcement is necessary only for cracked sections.

C26.9.2 Bridges designed under the allowable stresses of this specification should be uncracked at service load levels. Fatigue of prestressed reinforcement will not occur in uncracked sections due to the related small stress range. Fretting fatigue due to rubbing between duct and strand also does not occur in uncracked sections.

C26.10.1 See Paragraph 26.5.2.3(a) for allowable local zone bearing stresses under anchorage plates, and allowable general zone tensile stress in reinforcement for the anchorage splitting force. The Commentary to Paragraph 26.5.2.3(a) provides references for anchorage zone analysis and design. The pattern of splitting stresses due to bearing plate anchorages the same width as the web is illustrated by Figure C26.10.1. Note that the maximum splitting stress occurs at 1/4 d to 1/2 d in front of the anchor. The value of the total bursting force in Paragraph 26.10.2(a) is an approximation of the area under the splitting stress curve in Figure C26.10.1.

C26.10.2 The strut-and-tie approach suggested by Schlaich et al.[2] will give a good approximation of the reinforcement quantity and distribution required to counteract the general anchorage zone tensile forces set up both directly in advance of the anchorages (see Figure C26.8.4.1B) and in the outer regions of general anchorage zones with eccentrically located anchorages (see Figure C26.10.2). The anchorage local zone becomes a node for the strut-and-tie model and the adequacy of the node must be checked by appropriate analysis or full scale testing as required under Paragraph 25.5.2.3(b).

The center of the bursting force is located approximately 3/8 of the depth of the section in front of the anchorage (see Figure C26.10.1).

Tendon inclination, tendon curvature, and the blockout to achieve tendon inclination at the face of the anchorage all increase the bursting stresses.[29]

C26.10.3.2 Local anchorage zone reinforcement supplied as part of a proprietary post-tensioning system shall be shown on post-tensioning system shop drawings. Adjustment of general anchorage zone tensile reinforcement due to reinforcement supplied as part of a proprietary post-tensioning system may be considered as part of the shop drawing approval process. The responsibility for design of general anchor zone reinforcement remains with the Engineer of Record.

C26.10.3.3 For flange thickness ranging from 5 to 9 inches, an upper limit of 12½" ϕ 270 k strand is recommended for tendons anchored in blisters supported only by the flange. The anchorage force of 347 kips for a tendon of this size must be carefully distributed to the flange by reinforcement.

C26.11.2 Excess capacity may be provided by use of oversize ducts and oversize anchorage hardware at selected anchorage locations.

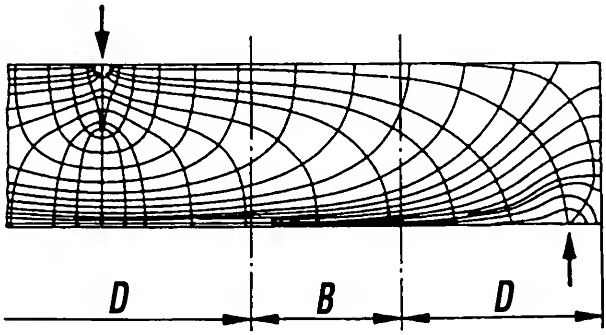


Figure C26.8.1A. Stress trajectories in a B-region and near discontinuities (D-regions).*

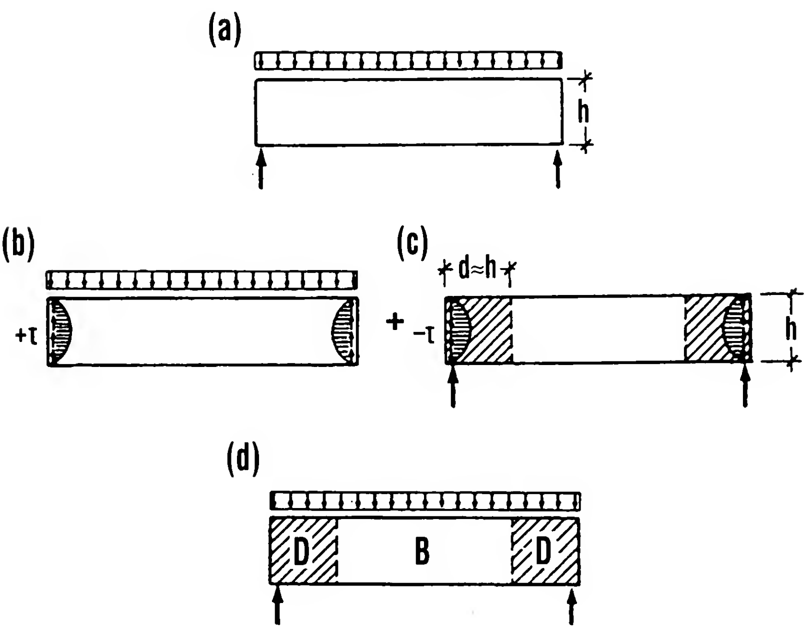


Figure C26.8.1B. Beam with direct supports.*

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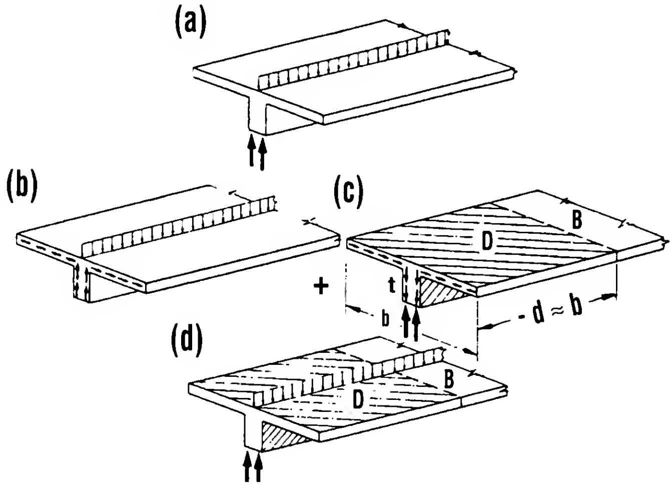

Figure C26.8.1C. T-beam: (a) real structure, (b) loads and reactions applied in accordance with Bernoulli hypothesis, (c) self equilibrating state of stress, and (d) real structure with B- and D- regions.*

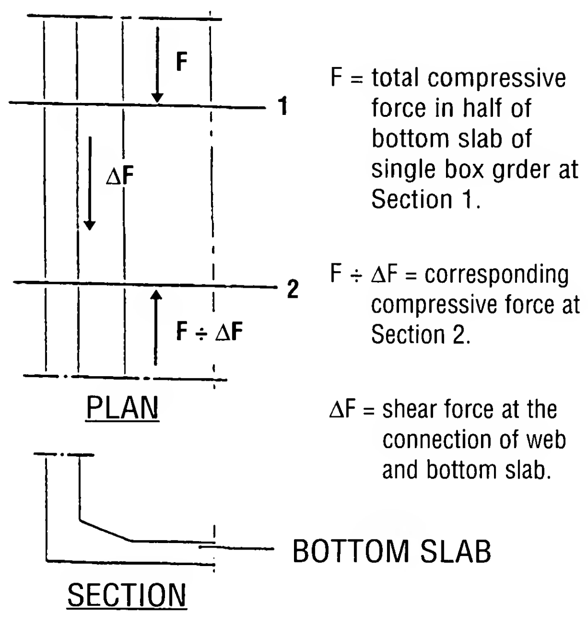

Figure C26.8.1D. Longitudinal shear transfer by bottom slab to web haunches.*

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C26.11.3 This provides for future addition if internal unbonded post-tensioning tendons draped from the top of the diaphragm at piers to the intersection of the web and bottom slab at midspan. Tendons from adjacent spans have to be lapped at opposite faces of the diaphragm to provide negative moment capacity. The requirement of a force of 5 percent of the total positive moment and negative moment post-tensioning force is an arbitrary value. Provision for larger amounts of post-tensioning might be developed as necessary to carry specific amounts of additional dead load as considered appropriate for the structure.

C26.12.1.1 Thickness of metal duct material is related to duct diameter and the method of installing the tendon. Strand tendons are normally installed in the duct after the concrete is placed, requiring a stiffer duct. Bar tendons are preassembled inside small diameter ducts and placed as a unit. The bar fills most of the void and helps to prevent duct damage. The use of epoxy coated metal duct is not recommended due to questionable bond characteristics.

C26.12.1.2 Ontario Ministry of Transportation tests indicate a tendency for air entrapment for ducts with concentric corrugations.

ASTM D 2239 relates to rigid pipe manufactured by a process based on controlled inside diameter. ASTM D 2447 and ASTM F 714 relate to rigid pipe manufactured by a process based on controlled outside diameter. All three specifications produce pipe satisfactory for bridge applications.

C26.12.2 Placement of tendons by the pull-through method requires duct area of 2½ times the prestressing steel area specified for grouting.

C26.12.3 Polyethylene duct abrades at curvature radii less than 30 ft.

C26.12.4.1 It is recommended that duct support requirements be stipulated or shown in the contract documents.

C26.12.4.2 External ducts are normally polyethylene.

C26.12.6.2 The hairpin bars tie the slab together in event of spalling forces at slab joints.

Ducts spaced closer than 12 in. on center in either direction should be considered as closely spaced. The hairpin bars are provided to prevent slab delamination along the plane of the post-tensioning ducts. The hairpin bars are not required in areas where duct congestion does not exist.

C26.13 European experience indicates that the prestressing force decreases locally in the region of a coupler. This is believed to result partially from increased creep caused by high compressive stresses in the reduced concrete Article due to coupling of tendons. Cracking has not been observed in bridges where the number of tendons coupled at an Article has been limited to 50 percent of the total number of tendons.

C26.14 Figure C26.14[20] provides schematic illustration of various methods of transmitting load from secondary beams to the main beam.

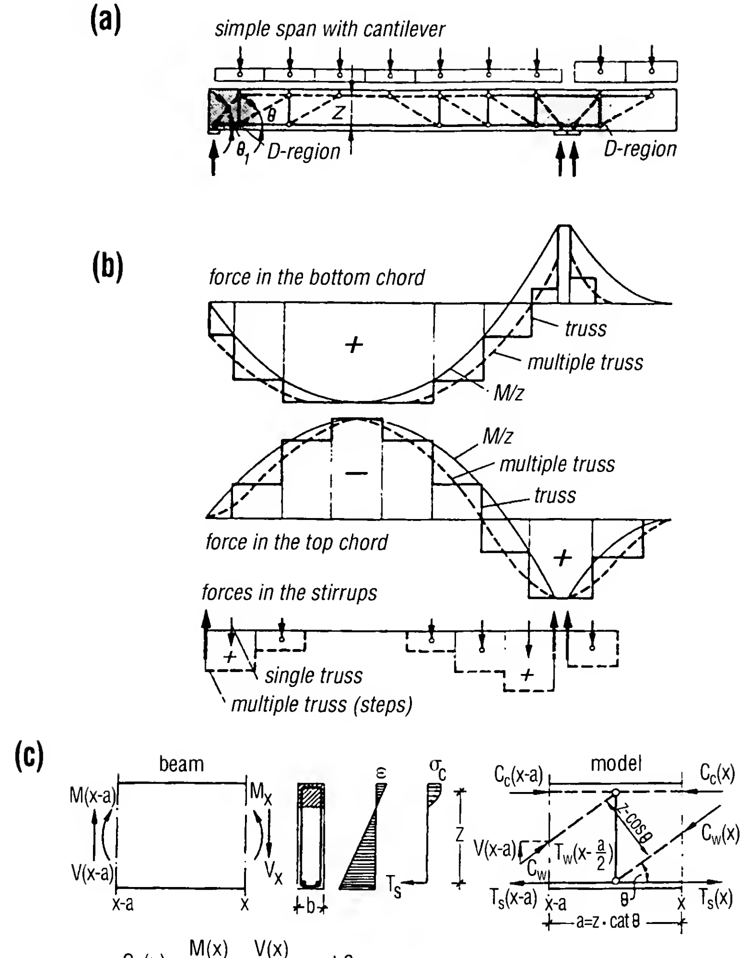
C26.15.2 The quantity Z provides reinforcement detailing that will reasonably control flexural cracking. Crack potentials are largest when handling and storing segments for precast construction and when stripping forms and supports from cast-in-place construction.

C26.17.1 The top flange thickness of 9 inches is preferable in the area of anchorages for transverse post-tensioning tendons.

Further research is recommended on the transfer length of 0.6 inch diameter strand before such strand is used for transverse pretensioning in thin sections of segmental bridges.

C26.17.2 Ribbed webs may be reduced to 7 in. thickness.

C26.17.4 Girder depth and web spacing determined in accordance with the following will generally provide satisfactory deflection behavior:



$$C_c(x) = \frac{M(x)}{z} - \frac{V(x)}{2} = \cot 8$$

$$C_w(x) = \frac{V(x)}{\sin 8} \rightarrow \quad \sigma_w(x) = \frac{V(x)}{b - z \sin 8 \cos 8} \text{ (smeared diagonal stress)}$$

$$T_w\left(x - \frac{a}{2}\right) = V(x) \rightarrow t_w\left(x - \frac{a}{2}\right) = \frac{V(x)}{z \cdot \cot 8} \text{ (per unit length of beam)}$$

$$T_s(x) = \frac{M(x)}{z} + \frac{V(x)}{2} \cot 8$$

$V(x)$ may include shear forces from torque M_T accordingto Figure 28

Figure C26.8.4A. Truss model of a beam with cantilever: (a) model; (b) distribution of inner forces; (c) magnitude of inner forces derived from equilibrium of a beam element.*

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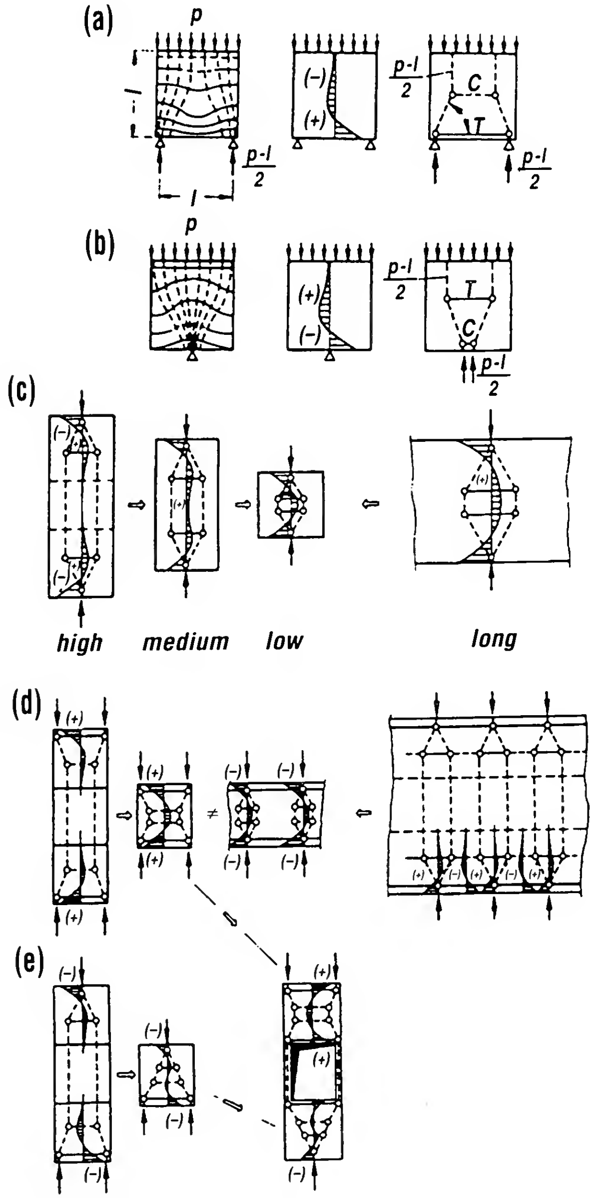

Figure C26.8.4B. The two most frequent and most useful strut-and-tie models: (a) through (b), and some of their variations (c) through (e).

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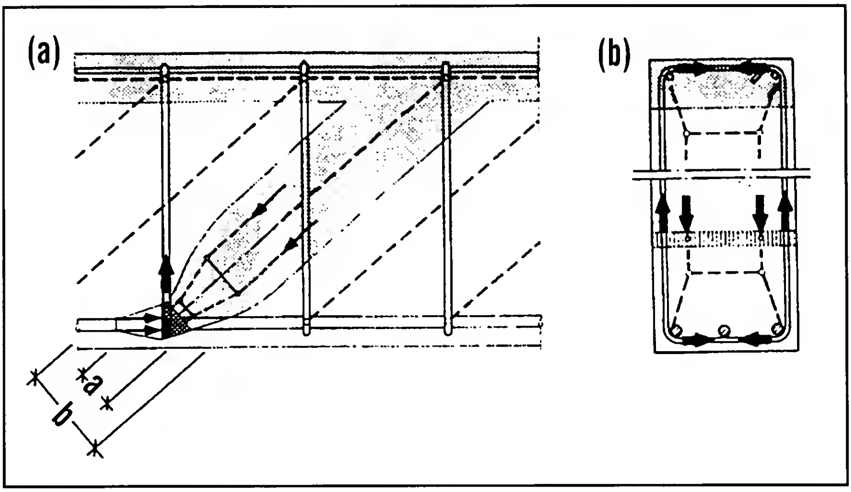

Figure C26.8.4C. The compression strut in the web with the stirrups.*

A. Constant depth girder

$^1/_{15} > d_o/L > {}^1/_{30}$
optimum $^1/_{18}$ to $^1/_{20}$

where d_o = girder depth, feet

L = span length between supports, feet

In case of incrementally launched girders, the girder depth should preferably be between the following limits:

$L = 100'$, $= {}^1/_{15} < d_o/L < {}^1/_{12}$
$L = 200'$, $= {}^1/_{13.5} < d_o/L < {}^1/_{11.5}$
$L = 300'$, $= {}^1/_{12} < d_o/L < {}^1/_{11}$

B. Variable Depth Girder with Straight Haunches

at pier $^1/_{16} > d_o/L > {}^1/_{2}$

optimum $^1/_{18}$

at center of span $^1/_{22} > d_o/L > {}^1/_{28}$

optimum $^1/_{24}$

(NOTE: a diaphragm will be required at the point where the bottom flange changes direction.)

C. Variable Depth Girder with Circular or Parabolic Haunches

at pier $^1/_{16} > d_o/L > {}^1/_{20}$

optimum $^1/_{18}$

at center of span $^1/_{30} > d_o/L > {}^1/_{50}$

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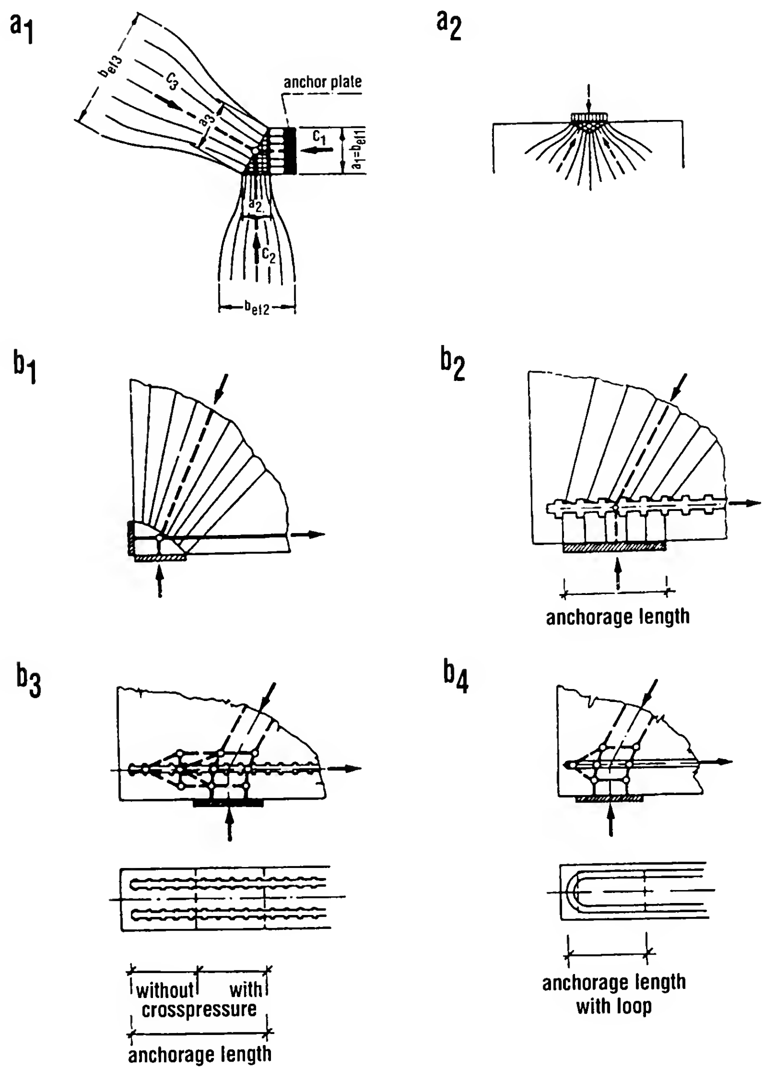

Figure C26.8.4D. Examples of the basic types of nodes; (a) CCC-nodes. Idealized "hydrostatic" singular nodes transfer the concentrated loads from an anchor plate (a) or bearing plate (a_1) into (bottle shaped) compression fields, (b) CCT-nodes. A diagonal compression strut and the vertical support reaction are balanced by reinforcement which is anchored by an anchor plate behind the node (b_1), bond within the node (b_2), bond within and behind the node (b_3), bond and radial pressure (b_4).*

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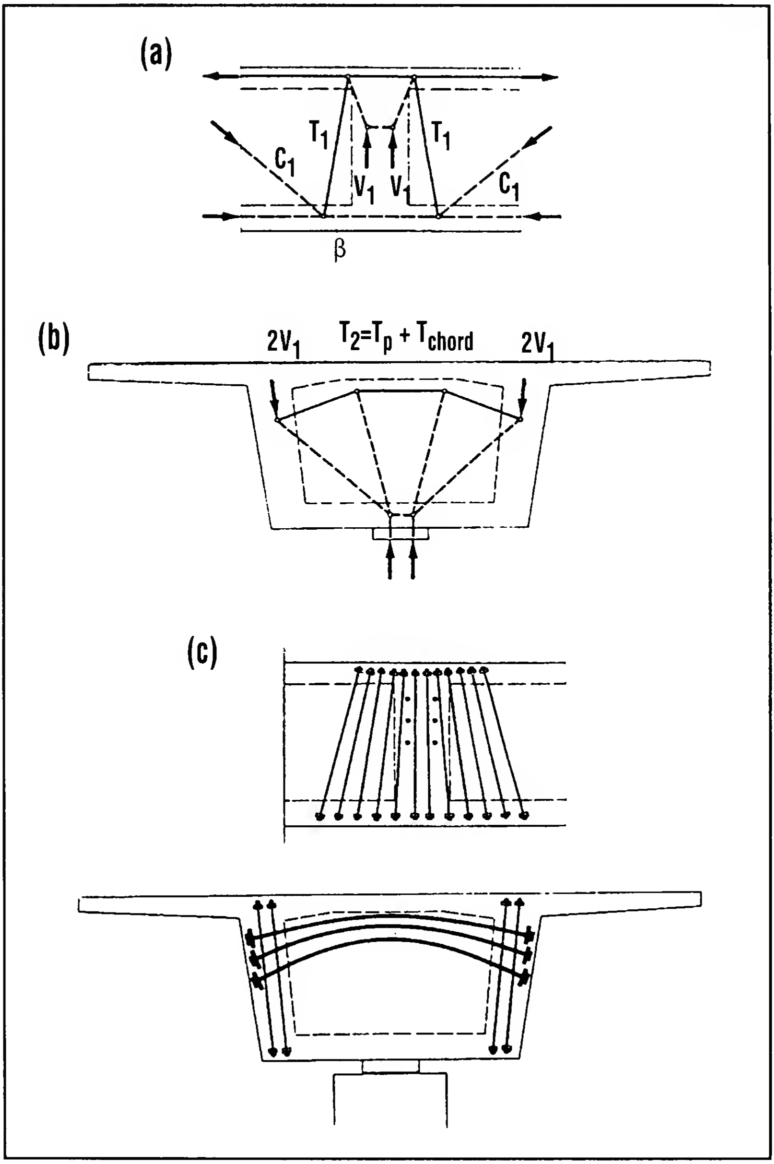

Figure C26.8.5.2A. Diaphragm of a box girder bridge: (a) D-regions and model of the web near the diaphragm; (b) diaphragm and model; (c) prestressing of the web and the diaphragm.*

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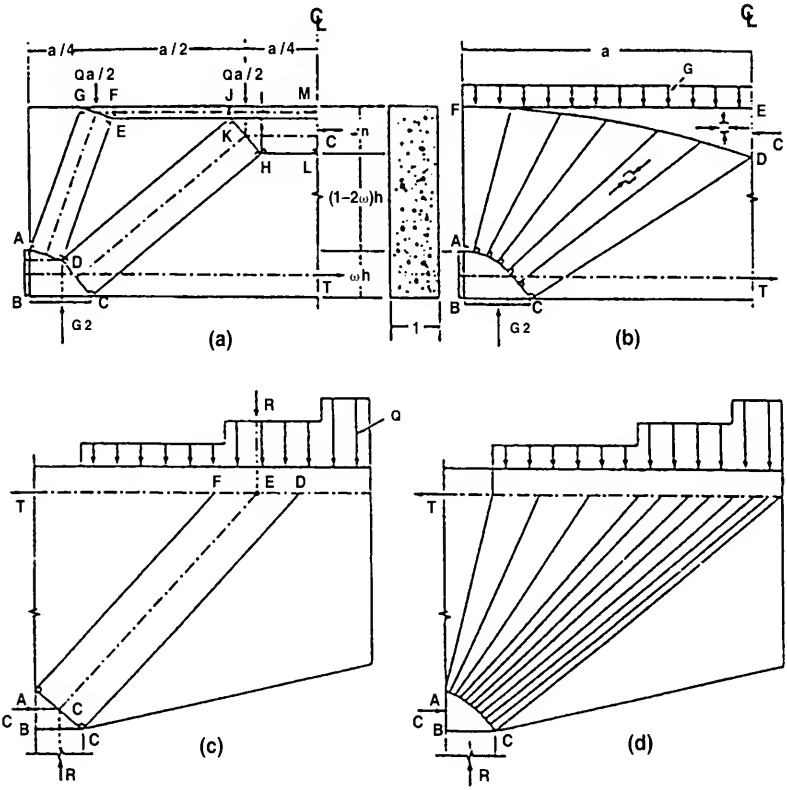

Figure C26.8.5.2B. Fan action: (a) Strut-and-tie model of uniformly loaded deep beam; (b) Fan-shaped stress field; (c) Strut-and-tie system for equivalent single load R replacing distributed load q; (d) Continuous fan developed from discrete strut.*

D. Depth to Width Ratio

A single cell box should preferably be used when

$$d_o/b \geq 1/6$$

A two cell box should preferably be used when

$$d_o/b < 1/6$$

where b = width of the top flange.

If in a single cell box the limit of depth to width ratio given above is exceeded, a more rigorous analysis is required and may require longitudinal edge beams at the tip of the cantilever to distribute loads acting on the cantilevers. An analysis for shear lag should be made in such cases. Transverse load distribution is not substantially increased by the use of three or more cells.

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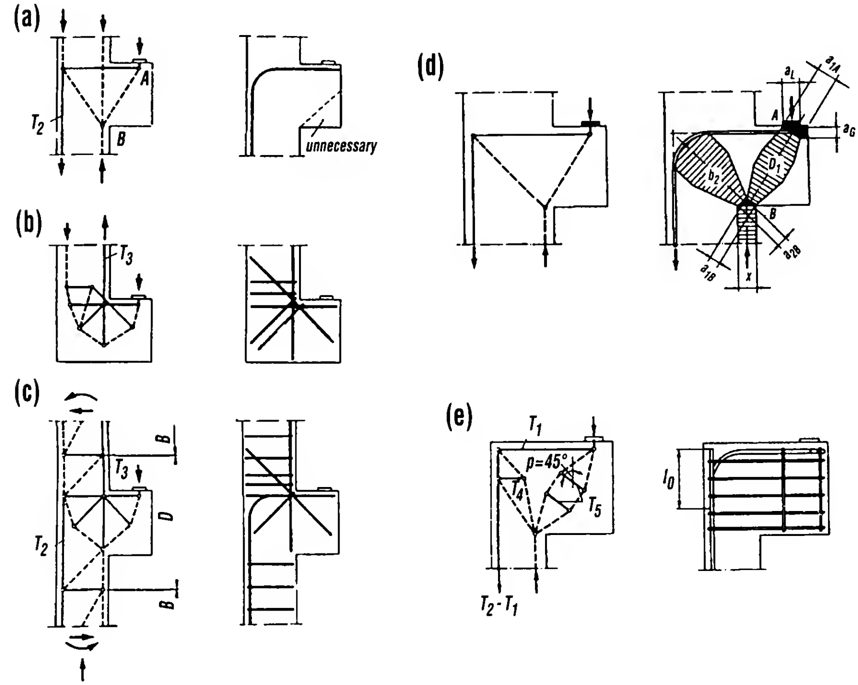

Figure C26.8.5.3. Different support conditions lead to different strut-and-tie models and different reinforcement arrangements of corbels.*

Part 26—References
Recommendations for the Design of Segmental Bridges

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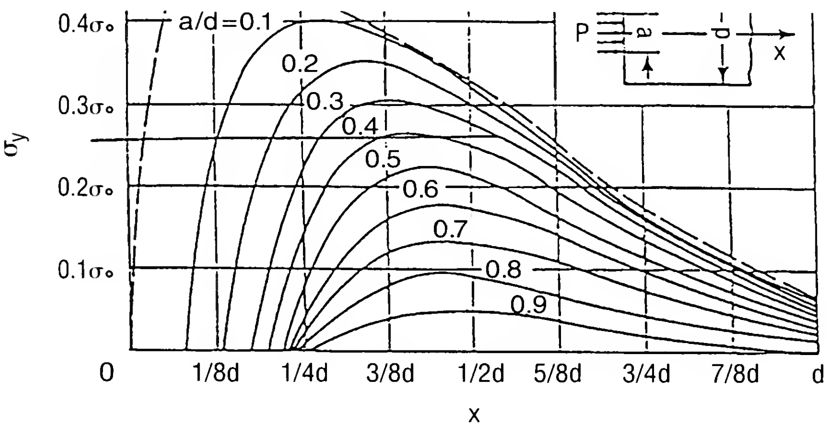

Figure C26.10.1. Bursting stresses under bearing plate anchorages.*

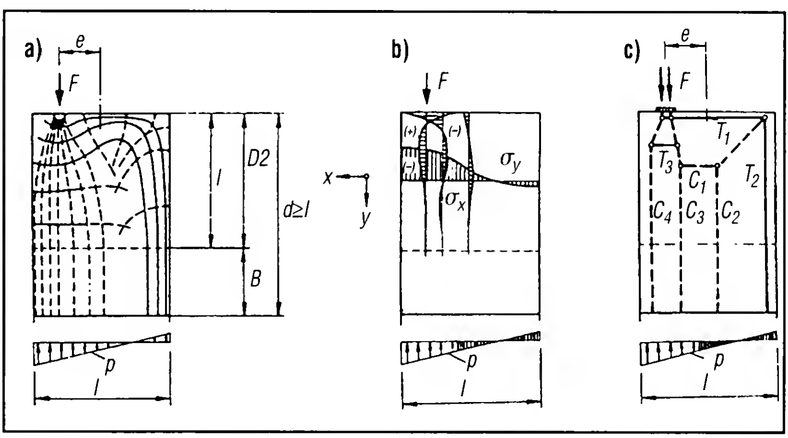

Figure C26.10.2. A typical D-region: (a) elastic trajectories; (b) elastic stresses; (c) strut-and-tie models.*

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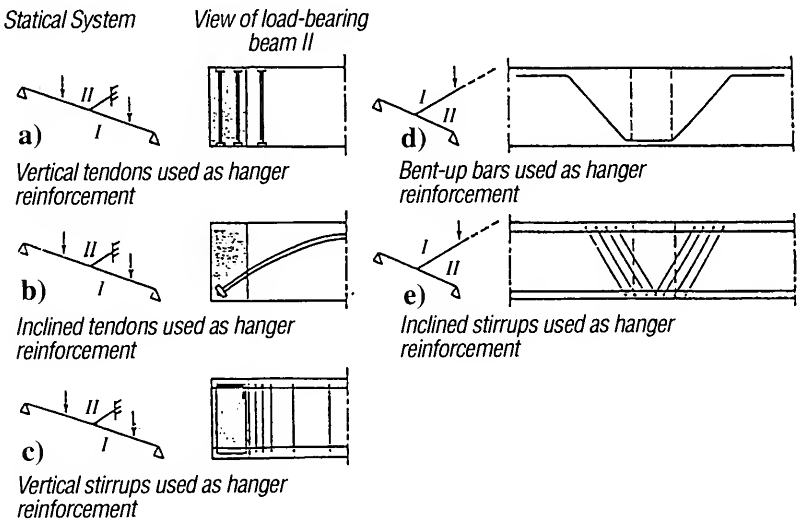

Figure C26.14. Schematic drawing of different types of "hanger" reinforcement.*

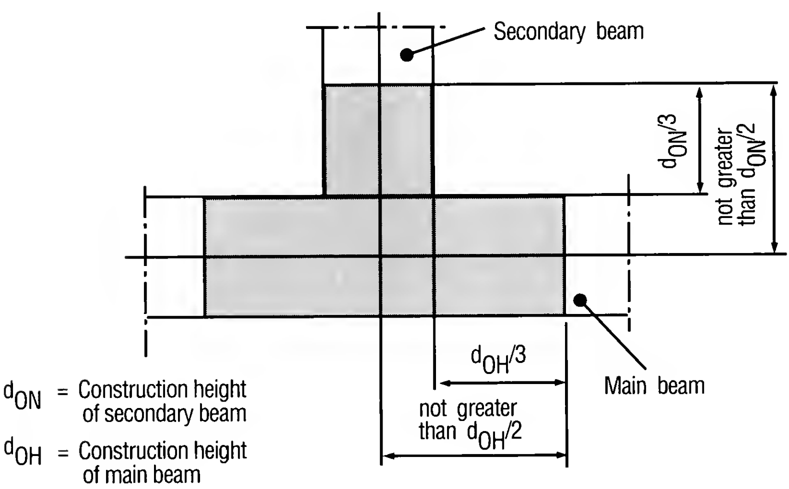

Figure C26.14A. Extent of the intersection zone for the connection of secondary beams.*

*Republished through the courtesy of the Prestressed Concrete Institute, *PCI Journal*, V. 32, No. 3, May–June 1987, pp. 74–150.

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Proposed 1996 Manual Revisions to Chapter 9—Seismic Design For Railway Structures

Page 9-1-15. Part 9.4 Design of Structures

Article 9.4.2, Paragraph (d), Item 3.

Delete the following sentence:

Allowable stresses may be increased by one-third for all combinations of loading which include seismic effects and increased 50% for structural steel.

Page 9-1-19. Part 9.5 Existing Structures

Article 9.5.7.2. Replace the current article with the following:

Timber trestles may be screened and eliminated from further evaluation if they are free of conditions that would require attention in the near future to permit continuation of normal railroad traffic. Seismic evaluation of timber trestles not eliminated by screening should focus on the potential effect of a seismic event on deficient conditions or details.

Proposed 1996 Manual Revisions to Chapter 10—Concrete Ties

Page 10-1-13. Part 1.2 Material

Article 1.2.4 Tie Pads. Replace the current article with the following:

Tie pads shall be used between the rail and concrete ties to minimize water intrusion and tie abrasion of the rail seat area and to reduce impact and vibration effects on the track structure. Recent experience has shown that dual durometer rubber pads, which have 50 to 60 Shore A durometer on the bottom surface and 75 to 85 Shore A durometer reinforced rubber on the top surface, have performed satisfactorily on tangents and curves up to 5 degrees. Alternate materials with similar characteristics may be used.

For curves over 5 degrees, a three-part sandwich pad consisting of lower layer foam gasket, intermediate layer metal abrasion plate, and top layer of tough thermoplastic material has been found to be satisfactory.

Article 1.2.4.1 Requirement

Change the first sentence of 2nd paragraph to read:

Tie pad shall have minimum width equal to the base width of the Rail (+ 1/8-0 inches).

Article 1.2.4.2 Material Tests. Change paragraph (a), compression set a low temp./ to –20°C. Eliminate note at end of paragraph (d), which reads "Note: eliminate resistance to ozone."

Page 10-1-31 Part 1.12 Ballast

Modify this entire section on ballast to read as follows:

1.12 BALLAST

1.12.1 Scope

Refer to Part 2, Chapter 1, for all ballast requirements.

Metrication

The entire Chapter 10 has been metricized.

Reproduced below are two (2) sample pages illustrating the metric conversion of Chapter 10. Copies of the entire chapter may be requested from AREA Headquarters.

Page 10-1-7. Article 1.1.2.5.1.1 Ballast Pressure*

Where:

P = Wheel load in pounds (kN).

IF = Impact factor in percent.

DF = Distribution factor in percent (from Figure 1.1.2.3.1).

A = Bearing area of cross ties in square inches (millimeters).

*For example: Given 8 ft. 6 in. long by 12 in. wide (259 cm long by 30 cm wide) concrete ties, what is the calculated value of bearing pressure for a locomotive with 30,000-lb (134 kN) wheel load if the ties are to be spaced at 28 inches (710 mm)

$$\text{Average Ballast Pressure psi (kPa)} = \frac{(2P)\left[1+\frac{IF}{100}\right]\left(\frac{DF}{100}\right)}{A}$$

$$= \frac{60,000\ (3.0)\ (0.56)}{102 \times 12}$$

$$= 82.4 \text{ psi } (0.586 \text{ MPa})$$

The recommended ballast pressure should not exceed 85 psi (0.586 MPa) for high-quality, abrasion resistant ballast. If lower quality ballast materials are used, the ballast pressure should be reduced accordingly.

1.1.2.5.1.2 Subgrade Pressure

The pressure exerted by ballast on the subgrade depends upon the tie-to-ballast pressure, the load distribution pattern through the ballast, and the depth of ballast. Refer to Section 1.12.

1.1.3 Lateral Loads

The lateral loads generated by moving railway equipment are applied by wheel treads and flanges to the rails, which in turn must be held in place by fastenings, ties and ballast.

Lateral stiffness of rail distributes lateral loads to fasteners and their ties. Structural strength of fastenings and ties hold the rail to gage. The mass of ties, friction between the ties and ballast, lateral bearing area of ties (end surface), and the mass of ballast all act to restrain lateral tie movement.

Lateral track stability can, therefore, be increased by decreasing tie spacing of ties of similar dimensions, increasing tie mass, increasing end bearing area of ties per unit length of track, and by increasing frictional resistance between ties and ballast. Structural strength of fastenings must be commensurate with the lateral load individual ties restrain, which in turn is determined by lateral rail stiffness and tie spacing.

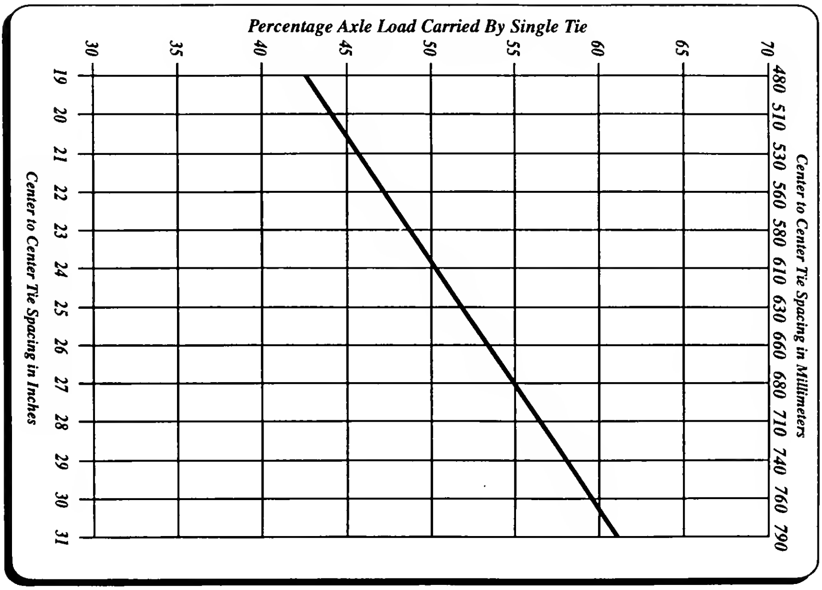


Figure 1.1.2.3.1

(end of sample pages)

Proposed 1996 Manual Revisions To Chapter 11—Engineering Records and Property Accounting

Part 2—Cartographic Specifications (Office & Drafting Practices)

Transfer this entire Part to new Part 11, Computer Aided Drafting—(CAD) of Chapter 32.

Retain reference in Chapter 11, as follows:

See Part 11 of Chapter 32 for Cartographic Specifications (Office & Drafting Procedures)

Proposed 1996 Manual Revisions To Chapter 12—Rail Transit

Part 3—Track and Roadway Consideration

Page 12-3-5 Add new article 3.2.1.2 Geometry, as follows:

3.2.1.2 Geometry

3.2.1.2.1 General

Wherever possible, track geometry is designed for optimum vehicle operating speed considering availability of right of way; patron safety and comfort; vehicle performance characteristics; station spacing; construction cost; maintenance considerations; and vertical and horizontal alignment.

Stationing is denoted throughout the route length along the centerline of the control track. In many cases of multiple track systems all tracks include stationing.

3.2.1.2.2 Horizontal Alignment

The horizontal alignment of main line tracks consists of tangents joined to circular curves by spiral transition curves. Spirals are also desirable in all heavily used tracks other than main line tracks, where practicable, between tangent and curves and between the different degrees of curvature of a compound curve.

Curvature and superelevation are related to design speed and the characteristics of the design vehicle.

3.2.1.2.3 Tangents

A desirable minimum tangent can be determined from the following formula:

$$L = 3V$$

Where:

L = Minimum tangent length, feet

V = Design velocity, mph

This minimum tangent distance allows patrons to recover from the effects of lateral force due to traversing a curve prior to entering another curve (which is often in the opposite direction). In many cases this distance may be difficult to achieve where right of way availability is limited. An absolute minimum length of tangent (usually not less than the length of the longest car to traverse the tangent) is required in order to prevent damage to vehicle couplers and articulation joints. The absolute minimum tangent length is determined based upon degree of curvature and vehicle/rolling stock requirements.

3.2.1.2.4 Circular Curves

Circular curves in many heavy rail transit (HRT) systems (i.e., rapid transit) are often defined by the arc definition and specified by their degree and/or radius as determined by the formulae found in Chapter 5, Track. Many commuter rail systems (which often share track with freight traffic) define curvature by the chord definition as outlined in Chapter 5, Track.

Curves on light rail transit (LRT) systems are usually defined by the arc definition and are specified by centerline radius and central angle (degrees-minutes-seconds). The definition of curvature for existing properties should be confirmed by the owner or operator of the system.

Allowable speed through circular curves is dependent upon degree of curvature, superelevation, track gauge and length of transition spirals. Design speeds are established such that given the parameters indicated above, the lateral (centrifugal) force will not exceed a specified value.

3.2.1.2.5 Spiral Transition Curves

Spiral transition curves provide a smooth transition from tangent track to curved track and should be used on all main line horizontal curves. Spiral transition curves can usually be omitted where the length for spiral (L_s) divided by the radius (R) using either feet or meters, is less than 0.01.

An acceptable spiral length can be determined from the equations given below in paragraphs a, b, or c. Each equation represents one of the factors which governs the length of the spiral. The maximum value calculated should be used as the minimum length of spiral. These formulae are valid only for standard gauge (4'–8½") track. Use of non-standard gauge track will require revised formulae.

a. Rate of Change of Superelevation (see 3.2.1.2.8), or Roll Rate

For speeds up to 60 mph a maximum rate of superelevation application of 1¼ inch per second is typically utilized, giving a formula: $L_s = 1.17\ E_aV$

For speeds in the 60–80 mph range a maximum rate of superelevation application of 1¹⁄₁₆ inch per second is typically utilized, giving a formula: $L_s = 1.26\ E_aV$

Another form of expressing the rate of superelevation change per second is as "Roll Rate" in radians per second or degrees per second. The above equations will usually govern spiral length where the actual superelevation is large or where there is little or no unbalanced superelevation.

b. Rate of Change of Lateral Acceleration

Based on typical values used for maximum lateral acceleration through the curve of between 0.10 and 0.12 g/sec and a rate of application of lateral acceleration between 0.03 and 0.04 g/sec^2, the spiral length will be within the values given by the formulae $L_s = 1.22E_uV$ or $L_s = 1.63\ E_uV$. When a large number of standing passengers is anticipated the maximum values for lateral acceleration and rate of application may be reduced to provide improved ride quality.

This spiral length equation usually governs for curves designed with maximum permissible unbalanced superelevation.

c. Vehicle Torsion

For North American Railways it has been determined that the front and rear diagonal corners of a car should not be warped more than two inches if the potential for derailment is to be avoided. For example, a car having a 62 foot wheelbase and allowing a 100% factor of safety, (i.e., one inch of warping) the track warp must not be more than one inch in 62 feet, giving the equation $L_s = 62E_a$. Articulated light rail vehicles (LRV's) must consider wheelbase from the center to the end trucks. A typical LRV may thus give the equation $L_s = 30E_a$. This spiral length usually governs for low speeds or curves with high superelevation.

Where:

L_s = Spiral length, feet

E_a = Actual superelevation, inches

E_u = Unbalanced superelevation, inches

V = Velocity, mph

Note that the factor of 62 or 30 relates to a transit vehicle with a standard three-piece truck. Many transit vehicles have an entirely different truck configuration. In any case the manufacturer's recommendations should be followed as to the allowable torsion (track twist) that the vehicle can tolerate.

3.2.1.2.6 Reverse Curves

Reverse curves should be avoided on main line track. If reverse curves are used, the minimum tangent length and spiral length as described in sections 3.2.1.2.3 and 3.2.1.2.5 should be used. An absolute minimum tangent length between reverse curves is governed by vehicle and/or rolling stock requirements.

If circumstances do not permit the minimum tangent length to be accommodated between reversing curves, the spiral transitions may meet at the point of reverse curvature, provided vehicle torsion considerations do not govern the spiral length. The track twist is doubled at back-to-back spirals and the spiral length should be doubled in these circumstances.

3.2.1.2.7 Compound Circular Curves

Compound curves should be avoided on main line track. If compound curves are used, a spiral should be inserted between the circular curves. The same considerations for minimum spiral length shall be met in these cases.

3.2.1.2.8 Superelevation

Superelevation is the height difference between the high (outside) and low (inside) rail on a curve which allows higher speeds. Superelevation should be constant throughout the entire length of the circular curve. The superelevation should increase linearly throughout the length of the spiral curve.

The track superelevation may be determined from the formulas indicated in Chapter 5, Track, or modified to suit specific transit applications. When developing a maximum allowable unbalanced superelevation on a new transit system, patron comfort and safety is a primary consideration. See Chapter 5, Track, for definition and derivation of unbalanced superelevation.

Superelevation is generally not used in yard track except in cases such as long "loop" tracks where superelevation may be desirable in order to reduce wear on the high rail.

3.2.1.2.9 Vertical Alignment

The vertical alignment consists of vertical grades connected by parabolic vertical curves having a constant rate of change in grade.

The vertical alignment is generally set at the top of the low rail (profile rail).

3.2.1.2.10 Vertical Grades

Vertical grade constraints are generally determined by the acceleration, deceleration and wheel/rail adhesion characteristics of the design vehicle. For efficiency and performance, all grades should be the lowest percent that is practical.

The maximum grade for HRT generally ranges from 3.0 to 4.0 percent. A typical desirable maximum grade is 3.0 percent.

The maximum grade for LRT generally ranges from 4.0 to 7.0 percent or greater. A typical desirable maximum grade is 4.0 percent.

There are a number of factors which govern vertical grades. The following is a partial listing of factors the designer should consider when selecting maximum and minimum grades:

— Acceleration characteristics of the vehicle. (Velocity profile calculations may be required).

— Braking capabilities of the vehicle.

— The presence of horizontal curves on grade (curve compensation calculations may be required).

— Grade limitations through stations.

— The possibility of increasing/decreasing grades to assist in vehicle acceleration or braking.

— Limitations due to sustained grades.

— Limitations to accommodate trackside drainage.

— Grade constraints for maintenance yards (i.e., avoidance of runaway vehicles).

— Limitations due to topography and existing features (as encountered in street running systems).

— Limitations for multi-use corridors (i.e., a joint commuter rail and LRT corridor).

3.2.1.2.11 Vertical Tangents

A minimum vertical tangent length is often desirable for patron comfort and/or vehicular constraints. A desirable minimum can be determined from the following formula:

$$L = 3V$$

Where:

L = Minimum tangent length, feet

V = Design velocity through tangent, mph

Some properties require an absolute minimum vertical tangent length of 100' (30 m) while others (street running systems) may allow a shorter vertical tangent length due to topographic constraints. When determining minimum vertical tangent lengths, vehicle characteristics must be considered.

3.2.1.2.12 Vertical Curves

All changes in grade are connected by parabolic vertical curves for smooth riding and appearance. The length of a vertical curve is determined by the grades to be connected, the design speed, and the design vehicle characteristics. A typical minimum vertical curve length is 100 feet. A preferred minimum is 200 feet. Vertical curves for LRT may be as low as 30 feet or less due to physical (topographic) constraints. Many LRT systems define minimum vertical curve lengths based on formulas derived from highway design (AASHTO) equations.

Vehicle manufacturers typically state physical constraints for vehicles in terms of combined horizontal and vertical curves as defined by radius. For this purpose, an equivalent radius for a given vertical curve can be approximated by using the following formula:

$$R_{EQ} = \frac{L}{|G1 - G2|}$$

Where:

R_{EQ} = Equivalent vertical radius, feet

L = Length of vertical curve, feet

G1 = Incoming grade, percent

G2 = Outgoing grade, percent

$|G1-G2|$ = Algabraic difference in grade, percent

Vertical curve lengths may also be limited by overhead contact wires (in electrified transit systems) due to the permissible rate of separation or convergence between the track and the contact wire. Applicable criteria for overhead contact systems should be considered.

3.2.1.2.13 Reverse Vertical Curves

Reverse vertical curves should be avoided on main line track. If reverse vertical curves are used, the minimum vertical tangent distance as described in section 3.2.1.2.11 should be used.

3.2.1.2.14 Special Trackwork

Special trackwork (turnouts, crossovers and track crossings) are normally installed within horizontal and vertical tangents. Exceptions are made in some cases because of restricted clearances or existing topographic conditions. See section 3.2.1.7, Special Trackwork, for additional information.

Part 5—Rail Transit Power Supply and Electrification Systems

5.1 INTRODUCTION

This section will offer guidelines to address the methods and procedures utilized in providing traction power and the control of traction power needed for the propulsion of Rapid Transit Systems.

5.1.1 References to Other Applicable Chapters (to be developed)

5.1.2 Approach to These Guidelines

These guidelines include descriptions, requirements, and functional design suggestions for the supply, distribution, and supervision of traction power systems for Light Rail Transit (LRT) or Heavy Rail Transit (HRT) Systems.

5.1.3 Scope and Definition of Electrification Systems

The following major components comprise design of the traction power system:

A. Alternating current or transformer-rectifier substations

B. Contact rail or overhead contact system

C. Running rails

D. Power cables

E. Raceway systems

F. Supervisory Control And Data Acquisition (SCADA)

The Transit Vehicles are propelled by electric traction motors driving steel wheels through the appropriate gearing. Electric traction power is normally supplied to the vehicle from wayside distribution through an overhead contact system distributing power through a contact wire installed over each running track, upon which a pantograph collector on each car maintains contact. For HRT, electric traction power is normally supplied to the vehicle from wayside distribution through a contact rail installed along each running track, upon which collector shoes or paddles on each car maintain contact. For LRT and HRT, the running rails of each track are used for the traction power negative return, except at crossovers or other locations as determined.

The traction power system must supply sufficient power to transit vehicles to provide safe, efficient, and continuous operation of the transit system. Design of the traction power system must be coordinated with the local electrical utility.

The traction power system design must be coordinated with other subsystems (vehicles, civil works, signaling, communications, SCADA, etc.) including vehicle propulsion (DC or AC) and power control operating tolerances. The vehicle auxiliaries should accept the full range of traction power system voltage variations.

5.1.4 Definitions and Glossary of Terms

Access-way
that portion of the ground, any floor, passage, stairway or other recognized fixed foothold, affording approach to high voltage electrical equipment, and on which it is intended that a person shall walk or stand while such electrical equipment is "alive"

ACSR
a stranded "Aluminum Conductor, Steel Reinforced," used for the messenger wire, and for return and feeder wires

Actual Span
span length as measured on the ground and not determined from the arithmetic difference of the two pole locations

Air Break Switch/Isolator
a switch or isolator, the contact of which make and break in the air

Alive
when an electrical conductor is at a potential different from that of the common return or any other conductor of the system of which it forms a part

Along Track Movement
the motion of catenary induced by counterweights and due to thermal expansion or contraction

Anchor
anchor bolt—a large bolt inserted into a drilled hole in rock or concrete and grouted to form a strong attachment

deadman—an anchor buried in ground, usually a rectangular block of concrete, to which a down guy is attached

plate—a buried plate at the end of an anchor rod used with down guys

rock anchor—an anchor used for down guys installed in holes drilled in rock which may have an expanding section at the end tightened into the rock by turning its anchor rod

screw anchor—a screw type blade at the end of an anchor rod used with down guys, installed by screwing into soil

Auxiliary Wire
an additional wire in a catenary system, installed between a messenger wire and the contact wire, used for high speed electrification

Berm
top edges on each side of an embankment

Block
see "Pulley"

Blow Off
see wind blow off

Bolted Base

a method of attaching foundations to poles and stanchions, by a bolting process

Bond

an electrical connection between metal hardware to eliminate static discharge

bond, impedance—a magnetic impedance device with center tap connected to grounding or return wire systems, itself connected across rails for signalization purposes; offers high resistance to A.C. current and almost no resistance to D.C. current

bond, rail—electrical connection between adjacent lengths of rail

Booster Transformer

a type of transformer with one winding in the catenary system, the other in the return wire circuit to absorb current from rails to the return wire; also known as suction transformer; not to be confused with auto-transformer used for raising voltage towards the receiving end of the line

Bracket

termed as Pole Bracket or Cantilever

fixing—the means by which the crossarm assembly is attached to the structure

swing mounting arm—the hardware used for attaching the crossarm to a pole which allows movement along the line by allowing the end away from the pole to swing

Bull Ring

a circular steel bar providing termination for multiple wires

Cable

a length of single insulated conductor (solid or stranded), or two or more such conductors, each provided with its own insulation, which are laid up together; the insulated conductor or conductors may or may not be provided with the overall mechanical and/or insulating protective covering

Camber

a preset defection curvature or the hog of a beam that maintains aesthetic appearance even after the beam is loaded

Cantilever

see "Bracket"

Carbon Collector

the carbon strip top of the pantograph or insert of a harp assembly on trolley poles which ride along the contact wire

Catenary

American use—the combination of conductors, hangers and in-span hardware of the overhead contact system, not including supports and crossarms

British use—the wire from which the contact wire is suspended by means of droppers or hangers, called "messenger wire"

compound catenary—comprising a contact wire supported from the auxiliary wire, which is supported from the messenger wire

dictionary meaning—1) the curve assumed by a perfectly flexible cord of uniform density and cross-section handing freely from two fixed points; 2) something in the form of a catenary

simple catenary—comprising a contact wire supported from the messenger wire *stitched*—an inverted Y or bridle arrangement, used at the supports of a catenary system to improve dynamic characteristics

trolley—comprising the contact wire only

Chainage
the stationing or mileage marking along an alignment or track

Chicago Grip
a tool for temporarily gripping and dead-ending wires under tension

Clamp

parallel clamp—a piece of hardware used to clamp two parallel wire together

pipe clamp—a piece of hardware used to attach various type of components to a pipe

strain clamp—a piece of hardware used for dead-ending a wire

suspension—a piece of hardware used to support a wire in hanging arrangement

Clearance Profile (or envelope), Open Line
the clearance envelope around a vehicle, pole, or contact wire system where it is clear of intrusion

Clipping In
sagging an overhead wire to correct tension and clamping it at the support and fixing hangers

Coffin Hoist or Chain Hoist
a construction tool used for pulling and adjusting wires under tension operated by a lever arm and a ratchet

Conductor
any body or substance specifically provided for the purpose of conducting an electric current

Constant Tension
constant tensioned conductors, normally provided by counterweights, spring tension, or by pneumatic and hydraulic tensioning devices

Contact Bridge
a rigid bar about 5 feet long fixed closely above the in-running contact wire forming a slot for a second contact wire to pass through

Contact Rail System
the positive electrical distribution system for transmission of traction power to the transit vehicles (third rail); an electrical conductor located alongside the track, designed to carry energy for the propulsion of trains

Contact Wire
the overhead wire with which the pantograph or current collector is designed to make contact, also called trolley wire

Contact Wire Height
the height of the underside of the contact wire above a road or rail level when not uplifted by the pantograph or trolley pole

Copperweld Conductor
a wire or a stranded cable made out of several wires, with a steel core and layer of copper fused around it

Counterbalance
another name for a counterweight (see "Counterweight")

Counterpoise
a buried wire or a configuration of wires to provide a low resistance to grounded systems, no covering as large an area as a ground mat

Counterweight
the weighted tensioning device at each end of the tension section, sometimes called counterbalance

Creep
the on-going permanent stretch of wires under tension for long periods of time

Crosby Clip
a wire rope clip

Crossarm
the support frame and registration assembly supporting the catenary

Crossovers
a casting for trolley contact system or in trackwork a track connecting parallel track (see "Frog")

Cross Span
indicates anything that crosses the overhead contact system or rails at catenary level

C.T.
indicates current transformers

Current Collector
see "Carbon Collector"

Curve
horizontal—curvature of a street or track

transition—a curve on increasing radius that connects from a curve to a tangent or another curve

vertical—the vertical curvature of the street road paving or the track

Curve Segment
see "Segments"

Dead
isolated and grounded

Dead-End/Fixed End
a tensioned conductor termination without automatic tension devices such as counter weights or springs

Disconnect
an off-load/"no-load" type electrical switch for disconnecting electrical power feed from a line section

Drop Bracket
an assembly fixed below a registration pipe that permits the heel of the steady arm to be adjusted for height

Dropper
British term for hanger

Earth, Earthing, etc.
British term for ground, grounding, etc.

Electrical Clearance
passing—air clearance between live parts of either the vehicle or the overhead contact system (OCS) and grounded parts of the fixed structures; it exists during the passage of the trolley, trailer, locomotive or car

static—air clearance between live parts of an overhead install-system structure and grounded parts of a fixed structure

Electrical Section
all streets or sections of railway lines provided with overhead contact system (OCS) equipment for electric traction purposes; term normally used in a catenary system indicating the distance between the lowest and highest energized conductor at a point of suspension (i.e., at a pole)

Encumbrance
see "System Height"

Envelope
a theoretical form which is greater than the actual item (see "Clearance Profile")

Equation
the mathematical relationship between two chainages along diverging at a given point—may be due to routes accumulating differential chainage at a point of reconvergence

Feeders
conductors which supply power to or augment the power-carrying capacity of the OCS conductors in a supply system

Field Drill
when necessary to drill a hole by the constructors

Fixed End/Dead-End
see "Dead-End"

Flange
the flat sides of an "I" beam or the edge of a railroad wheel extending outward from the tread area

Floating
a section of insulated wire with no proper electrical connections to a power supply or to a grounding circuit

Foundation

side bearing—a laterally loaded foundation (i.e., pier type)

spread—a foundation with a predominantly vertical load

Frog

a device where two running rails intersect and which provides flangeways to permit wheels and wheel flanges on either rail to cross the other; also an overhead piece of hardware providing the same function for trolley contact wires

Gage or Gauge

loading—the envelope around a track or road vehicle within which all loaded vehicles must remain while static or in motion

track—the distance between the inside running edges of a pair of rails of a track system at a particular distance below the head of the rail

Gantry

usually applied to cranes having two vertical legs with horizontal members supporting the lifting mechanism or a similar structure supporting equipment across railroad tracks

Gradient

contact wire—the average slope of the contact wire between two supports

track/pavement—slope of the track or pavement longitudinally in respect to level

Ground

the conducting mass of the earth

Ground Connection

a conductor installed or applied (in terms of these instructions) to ensure that electrical apparatus or components are grounded

Ground Mat

a buried grid covering a fairly large area for substations and power plants where low grounding resistances are required

Ground Rod

a metal rod with ground wire connection to disperse currents into the ground for safety

Ground Wire

the conductor installed for the purpose of providing electrical continuity between the supporting structures of the overhead contact system or transmission lines and the common return of the system or grounding system

Grounded

connected to the common return of the system and/or the conducting mass of the earth in such a manner as will ensure at all times an immediate discharge of electrical energy without danger

Grounded Drop Vertical

the rigid member fixed under the beam of a portal to which the crossarm is attached

Grounding

the act or operation of applying a ground connection

Grout or Levelling Grout
a fine cement mixture used for base plates at foundations

Guy
a steady or positioning wire

down or back guy—a wire attached high on a pole and coming down at an angle to an anchor

Hanger
a fitting providing vertical support connection by means of which the contact wire is suspended from messenger or auxiliary wire or bracket

auxiliary—the support between auxiliary and contact wires

Harp
a trolley pole collector shoe assembly

Head Span Wire
a wire between two points to support OCS but not anchored to ground

Heel of Steady Arm
the end opposite the contact wire clamp

High Voltage (h.v.)
a voltage normally exceeding 600 volts

Hi-Rail Equipment
rubber tire mounted cranes, trucks, etc. with added flanged steel wheels that can be operated on tracks or on roadways

Hog
to arch upward in the middle (opposite of sag)

Horns
the bent or angled downwards portion of the pantograph at either end of the carbon collector

Impedance Bond
an electrical bond between rails that has a high reactance to high frequency currents

Insulated Joint
a rail joint in which electrical insulation is provided between adjoining rails

Insulating Neutral Section or Isolating Neutral Section
see "Phase Break"

Insulation/Insulator
any body or substance provided and designed for the purpose of surrounding or supporting a conductor so as to restrict the flow of electricity to a desired path

disc—a bell-shaped insulator of glass or porcelain used singly or in strings

Isolate
to disconnect from all sources of supply

Johnnie Ball
usually a ceramic interlinking insulator in guy application

pin insulator—an insulator, usually upright, fastened by a pin, not suspended

spool insulator—glass fiber (spun) strain

standoff insulator—a solid core insulator with structural strength to use in tension, bending or compression situations, usually of porcelain over a fiberglass core unit

stick insulator—strain insulator—fiberglass; reinforced epoxy

strain insulator—an insulator or a string of disc, dirigo or fiberglass insulators used in line with a tensile loading capacity b a conductor in a horizontal position

strut insulator—same as "standoff insulator"

suspension insulator—an insulator or string of discs which are suspended in vertical position

synthetic insulator—an insulator made from fiberglass and plastic or some epoxy resin matrix for skirts

Jumper

generally an internal electrical connection in the overhead contact system, a short conductor installed to provide electrical continuity

full current jumper—a jumper of equivalent capacity as the OCS

overlap jumper—a full current jumper at tensioning overlaps

potential equalizer—a light internal jumper in the OCS to elevate components to similar voltage levels

trolley jumper—jumper between contact wire normally support from two adjacent crossarms

Keeper—Keeper Piece

a locking piece

Knuckle

a short rigid bar interconnecting transferring forces and stabilizing adjacent conductors

Lattice Structure

a structure built up from standard steel sections as primary and secondary members

Level Crossing

a British term for grade crossing; a road, walkway or railway crossing the track at rail level for motor vehicles, rail vehicles, animals or pedestrians

Live

an electrically energized circuit (see "Alive")

Loop

a secondary bypass systems installation to a main

Low Voltage (l.v.)

a voltage normally not exceeding 600 volts D.C. or an energized circuit potential below normal voltage level

Lug

crimped—an attachment to the end of a wire for an electrical connection using a gripping or crimping tool

terminal—a crimped or a soldered piece to terminate a wire for electrical connection

Mast
a pole of timber or metal erected vertically as part of a derrick or other structure to act as a support

Maximum Sag
the sag of conductors based o either the maximum temperature or a given radial ice loading, whichever is the greater, in accordance with local conditions

Messenger
the catenary wire from which the contact wire is suspended by means of hangers or a guy with carrier lashed cables

Midpoint (Anchor) Structure
the structure between two counterweights at which a constant-tension catenary is fixed

Midpoint Guy
the span guy that provides the midpoint restraint

Midpoint Guy Pole
the pole, normally with a down guy, that takes the strain from the midspan guy

Midspan Offset
the deviation of the contact wire from a curved alignment or the superelevated centerline of track at midspan

Neutral Section
see "Phase Break"

No-Bo
a contact wire insulator or sectioning device for trolley or LRV pantograph systems

O.C.B.
indicates an Oil Circuit Breaker in a substation

Offset
deviation of the contact wire from a curved alignment or the superelevated centerline of the track

Offset Pole
the pole next to the counterweight structure within an overlap section that carries the radial load of the anchoring equipment

Open Overlap Span
that portion of the overhead track equipment between two main structures, where the contact and catenary wires of two adjoining sections overlap and terminate, and where an electrical break between those sections can be effected by means of switching operations

Out-of-Running Equipment
a catenary that does not provide primary passage for the pantograph (see "In-Running")

Overhead Contact System (OCS)
that part of the overhead equipment comprising of the contact or trolley wire, catenary, supports, foundations, counterweights or tensioning devices and other equipment and assemblies that delivers electric power from the substation to the light rail or heavy rail vehicles

Overhead Line Track Equipment
all equipment included in the circuit between substation positive and negative feeder cable terminals and all support work provided specifically for supporting such equipment, but excluding tie-stations, tie-stations feeder cables and all track equipment feeders erected on independent structures

Overlap Pole
the structure which positions the two contact wires in parallel within an overlap section

Overlap Section
the portion of the overhead contact system between two main structures, where the contact and catenary wires of two adjoining sections overlap and terminate

Pantograph
the current collector apparatus mounted on top of a light rail or heavy rail vehicle

Pantograph Sway
lateral displacement of the pantograph induced by vehicle roll and lateral shock loads

Pantograph Up-Thrust
the nominal upward force exerted by the pantograph on the overhead wire

Periscope Bracket
a clevis type bracket to hold a small pin insulator and used on contact wire only systems

Phase Break
an arrangement located between two sections of the overhead contact system fed from different phases under which a pantograph may pass without shorting the phases

Pole Brackets
see "Bracket"

Portal
a passage for entry as into a railroad or tunnel

Presag
the static difference between the average contact wire height at the end droppers in a span and the height at midspan

Prestress
to apply to a ductile alloy conductor a higher tension than for normal to accelerate creep effect; the period may vary from 15 minutes to 48 hours according to requirements

Prestretch
same as "Prestress"

Pull Off/Push Off
the registration of a contact system towards or away from centerline for pull or track in relation to the pole

Pull Over
see "Steady Arm"

Pulley System
the combination of block(s) and sheave(s) and its attachment(s) used in messenger support and catenary termination systems

Radial Load
transverse or across-track horizontal loads induced by conductors due to deviation from a projected centerline

Rail Bond
a conductor connection between sections of rail

Rake
a pre-set lean of the pole from vertical above a foundation

Registration
the process of lateral restraint of conductors

Registration Pipe
the horizontal pipe to which the drop bracket or heel of the steady arm is attached

Return Wire
a conductor, normally mounted directly on the rail that provides a low impedance return to the substation

Ropelay
a type of extra flexible stranded wire conductor made of a large number of fine wires

Ruling Span
a weighted average span of a given section used in sagtension calculations

Saddle
the fitting supporting a wire in a post insulator or a hanger or dropper on a span or messenger wire

Sag
the vertical deviation of a conductor between two supports or the process of wire tensioning when employing the magnitude of sag measurement as a means to obtain a preset tension in a conductor

Section
the electrical circuit beyond one or between two or more switching points

Section Break or Gap
an electrical break in the overhead contact system permitting isolation of a section

Section Insulator
a device for dividing a contact wire or catenary system into electrical sections which maintaining mechanical continuity and continuous path for trolley poles or pantograph collectors

Sectioning
the dividing of an electrical distribution system or network into electrical sections

Segments
a curve segment is a device employed in special works to provide a smooth circular path for trolleybus collectors

Shield Wire
a wire, often galvanized steel, strung over a conductor system for lightning protection

Signal Rail
the track rail used as the conductor for the track circuits controlling signalling appliances, and not used for traction return current

Signalling
a low voltage system used for train operation signals

Skirt
the protruding discs around the core of an insulator

Sleeve
chaffing—a sleeve around a conductor to reduce damage from abrasion

foundation—a concrete circular tube placed in a drilled hole to provide a foundation; occasionally refers to a smaller diameter pipe fitted into a larger diameter pipe

repair—a sleeve around a conductor to repair local electrical damage

Snub
to dead-end fix or fix temporarily

Spacers/Adjustable
an adjustable spacer is employed on trolley wire assemblies to maintain separation of parallel contact wires and fitting

Span Length
distance along an alignment or track between supporting structures

Span Width
distance across an alignment or track between the masts of a portal, cross-span or headspan

Spiral
see "Curve Transition"

Sprawl
the stringline of the contact wire on inclined catenary

Stagger
the offset of the contact wire from a projected centerline at a support due to registration

Stagger Effect
contact wire deviation from a projected centerline due to the combination of unequal staggers and wind

Static Position of Wire
the position of the wire without uplift and without wind

Stay
a short rod or wire providing restraint; a guy

Steady Arm
the lateral restrainer on the contact wire at a structure also known as pull over

Stringing
installation of overhead wires

Stringing Blocks
sheaves used in pulling wires during stringing with one cheek that can be opened for inserting wires and pull lines

Stringline
the distance between the centerline of track arc and its chord at midspan

Switch

electric track—a trolley wire switch providing alternate path for trolleybus collectors electrically actuated

feeder—a switch for feeding or sectioning electrical circuits

frogs—a trolley wire switch or crossing for trolleybus collectors

isolating—an air-break switch provided for isolating and grounding electrical apparatus and circuits

System Height

the distance between messenger and contact wire of a catenary system normally at the support structure, also known as encumbrance

Tail Wire

tail guy—the wire that with an insulator joins the yoke plate to the counterweight assembly

Tangent

the straight portion of the alignment or track between curves or spirals

Tangling

the hazard to trolley poles and pantograph where another conductor is in lateral proximity to the inrunning contact

Tension Section

tension length—the length of contact system between its mechanical terminations

Tensioning

the method of controlling sag from supported wires by pull or weight

Thimble

closed—the loop with its two ends close together

open—a loop to wrap wires around with the two ends spread apart

T.O.R.

signifies "top of rail" used for measuring vertical heights

Track Gauge

distance between the inside running edge of the rails of a track system

Track Tolerance

cross level—allowable variation between height of each rail

lateral—allowable variation in the track alignment

vertical—allowable variation in track height

Transition

see "Curve Transition"

Traveller

see "Stringing Blocks"

Trolley

a current collector using a pole with a wheel making contact with the feeder wire rather than a sliding shoe or a pantograph

Trolley Wire
see "Catenary," "Trolley" (term sometimes used for contact wire)

Turnout
a track arrangement consisting of a switch and a frog with connecting rails by which engines and cars can pass from one track to another

Uplift
dynamic—lift of the conductor due to the passing of trolley poles and pantographs

static—lift of the conductor due to the stationary trolley poles and pantographs

Vertical Reference Point
that point on centerline exactly midway between the two elevations where there is superelevation

Warning Portal
a portal placed at level crossings to warn traffic of the high voltage overhead wire danger

Web
the connecting area between two flanges of a steel beam or guides; the area of a steel rail between the head and base

Wind Blow-Off or Throw-Off
lateral movement of wires due to wind pressure

Yoke
a plate or casting whereby two or more wires or insulators terminate on one side and continue with one wire from the opposite side—usually yoke-shaped and with lever action to distribute loads from a counterweight as required

5.2 ELECTRIC TRACTION SYSTEMS CONSIDERATIONS

5.2.1 Systems Selection and Design

Light rail transit usually operates single cars in multiple consists of up to four cars or more. Light rail transit is suited for street running modes and on semi-exclusive or exclusive ROW. LRT operating in street running modes obey traffic signals along with motorists. Signal priority or preemption can be established with local DOT's to enhance train operations. Power for Light Rail Transit is usually supplied by an overhead contact system and collected through a pantograph on the car.

Heavy rail transit usually operates on exclusive ROW with two car units and in consists of up to 10 units. Power for heavy rail transit is usually supplied by contact rail or "third rail" and collected on the vehicle through collector shoes or paddles which ride on top of, or underneath, the contact rail.

The system should be designed to use proven hardware and design concepts. The systems fixed facilities (structures and buildings) should be designed for continued operation over a minimum period of 50 years. Major fixed system equipment should be designed for a minimum of 30 years.

Major equipment should be supplied by established manufacturers, have a documented operating history of previous and current usage, and be available off the shelf, so far as practicable. The same requirements should apply to spare parts.

Specifications for the system should be prepared in such a way as to encourage competitive bidding by established manufacturers of rail transit power supply and electrification systems.

5.2.2 Vehicle Propulsion Types and Performance

A. Direct current series motors geared to each or both axles of a truck.

Speed control is obtained by varying armature current and field strength. The current and torque produced at a standstill may be reduced by strengthening the field or lowering the terminal voltage or both. With the DC compound wound motor the start may be made with full armature current in the series field coils and maximum current in the shunt-field coils. Speed increase is accomplished by reducing resistance in the armature circuits until the armature and series field are connected across the line. Further speed is obtained by reducing the shunt-field current until the shunt field is disconnected.

Series-parallel operation of traction motors provides efficiency at low speeds with high tractive effort. Pulsating current motors and chopper control are refinements to speed control of the DC motor.

On 3000 Vdc systems usually two or three motors are operated permanently in series.

B. Alternating Current induction motors driven by power from two independent invertor systems, each driving two AC motor-gear units in two end car trucks. This system allows operation of single truck in the event of a malfunction of one invertor.

Advantages of AC propulsion systems are:

- Reduced maintenance
  With AC induction motors, brushes and commutators are not used.
- High efficiency
  Traction inverters use state-of-the art gate turnoff thyristors in combination with microcomputer control. Voltage and frequency are continuously controlled to maximize efficiency throughout the speed range for both motoring and braking.
- High reliability
  Bi-directional motion in motoring and dynamic braking without the use of customary power switchgear such as motor reversers, motor/brake setup switches, and field shunt contactors associated with DC traction drives are eliminated.

C. Single-phase commutator series motors

AC trolley voltage is stepped down to values suitable for direct application to the traction motors.

D. Asynchronous motors

Polyphase induction motors can be operated from 3-phase supply and from single-phase trolley circuit by use of phase converter mounted on the locomotive. By simultaneously varying the voltage and frequency, motor torque is closely regulated for precise speed control.

5.2.3 Operational Requirements

The following should be included in the traction power system:

1. Overhead Contact System (OCS) or contact rail system required for supplying power to the vehicles, including the catenary system, contact rail system, physical support system, and the associated feeder system.
2. Substations required for the use of AC or rectification of AC and supply of traction and facilities power, including high-voltage switching, protection apparatus, power transformers, rectifier transformers, rectifier assemblies, DC switchgear, positive bus, negative bus panels, interconnecting bus work, feeders, grounding and bonding system, control batteries and chargers, local and remote control, and supervision of equipment.
3. Electrical supervisory control interface functions required for supervising and controlling the performance of power and electro-mechanical equipment vital to the continuous operation of

the transit system. The traction remote supervisory control and indication functions should interface with the Supervisory Control And Data Acquisition (SCADA) system.

5.2.4 Adopted Electrification System AC or DC

Rapid transit systems usually operate on direct current systems supplied through transformer-rectifier equipment. Nominal voltages are in the range of 600 Vdc to 1500 Vdc.

5.2.5 Electrification Parameters

5.2.5.1 Climatic and Geographical Conditions

Design Environment

Environmental considerations for the Traction Power system to operate satisfactorily should include but not be limited to the following:

- Elevation
- Humidity
- Precipitation
- Ambient temperature:
  Highest recorded
  Lowest recorded
  Yearly average
- Conductor Design Temperature Range: (bare copper wire, steel/aluminum rail)
  High
  Normal
  Low
- Atmospheric pollution
- Ice loading
- Lightning (Isokeraunic Level)
- Seismic Zone
- Soil
- Soil resistivity
- Winds

5.2.5.2 Traction Voltage

Traction voltage must be of sufficient potential so as to optimize the size of the traction power current carrying conductors to the nominal operating currents required of the train vehicles.

5.2.5.3 Type of Substation

Substation types vary from AC to DC and the rectification equipment can be of various state of the art designs. Transformer-rectifier DC substations in the voltage range of 600 Vdc to 1500 Vdc are most commonly used in the United States.

5.2.5.4 Grounding Philosophy

As a principal goal, the Traction Power system must be designed to provide safety both to personnel and to the overall system. The design of the grounding system must preclude any unsafe condition to either the system, personnel, patrons or the community.

DC systems are normally operated ungrounded, AC systems are grounded.

5.2.5.5 Voltage Ranges

Traction Power voltages can be in the range of 600 Vdc to 3000 Vdc or from 11 KVac to 50 KVac.

5.2.5.6 Power Distribution Systems

Electrification of Rapid Transit Systems has been accomplished by means of three power systems:

1. Alternating-current single-phase systems
2. Alternating current three-phase systems
3. Direct current systems

Direct current systems in the voltage range of 600 Vdc to 1500 Vdc are the most commonly adopted rapid transit systems for LRT or HRT.

Control of DC Power Supply

A. Power supply to contact rail or OCS should be controlled by DC feeder circuit breakers installed at rectifier substations. The power supply should be sectionalized so that a power section for each track portion between rectifier substations is supplied from both ends by DC feeder circuit breakers located at adjacent substations. Each circuit breaker should be equipped for automatic operation on a current "magnitude of increment or rate of rise" basis, or other apparatus to positively distinguish between heavy loads and faults, with a load-sensing automatic reclosure feature. In addition, wayside motorized dc disconnect switches should be installed at strategic locations such as crossovers and stations without traction power substations to provide further sectionalization and to enhance flexibility of train operations.

B. Circuit breakers and wayside sectionalizing switches controlling power zones should normally be operated remotely from Operation Control Center by means of the supervisory control system, preferably SCADA. Controls should be provided to permit local operation of all circuit breakers within substations.

C. Provisions should be made to permit interruption in power supply to power sections by means of emergency trip sections located at designated intervals throughout the system.

5.2.5.7 Substation Spacing

Basis for Location, Spacing and Rating

Traction Power Substations (TPSSs) should be located, whenever practical, at or near passenger stations to minimize voltage drops in contact rails or OCS and running rails during train acceleration. DC traction power Substations are typically located 1 to 1/4 miles apart. These locations should be optimized with respect to system safety, system efficiency, system availability, stray current control, and minimum life cycle costs.

Normal Operation

The spacing and rating of TPSSs must be designed so that adequate power will be supplied to the system, with all substations operating, to maintain rated train operating performance during peak-hour traffic conditions. This includes providing full performance train voltage levels to allow simultaneous starting of maximum number of design trains at any passenger station and at a reduced performance at any point between stations.

Contingency Operation

With any one TPSS out-of-service, one train of maximum design consist should be capable of starting and accelerating at rated train operating performance as if the system was operating normally. Two trains, however, should be able to start simultaneously at the outage but then may accelerate and perform at a reduced operating level.

5.3 ELECTRIFICATION SYSTEM DESIGN CHARACTERISTICS

5.3.1 Clearances (OCS)

Electrical clearances between the OCS and other facilities must be in accordance with State and/or local regulations.

Mechanical clearances between the OCS and other facilities must be in accordance with State and/or local regulations.

For vehicle-related clearances, full allowance should be included for dynamic displacement of the vehicle under operating conditions (including track and other installation and maintenance tolerances).

The following clearances should be maintained between live conductors (including pantograph) and any grounded fixed structures in accordance with the AREA Manual (Chapter 33, Part 2) as follows:

	Passing	Static
Normal	4"	6"
Absolute Minimum	3"	5"

Passing clearance is the clearance between the catenary system or pantograph and an overhead structure under actual operating conditions during the short time it takes the power unit(s) of a train to pass.

Static clearance is the clearance between the catenary system when not subject to pantograph pressure, and the overhead structure.

Construction and Maintenance Tolerance

Design of the OCS should be based upon a total construction-plus-maintenance tolerance for the lateral and vertical locations of the structures as shown below.

Contact Wire

Location	Lateral	Vertical
Railroad Crossings	± 2"	± 1"
Overlap locations (between Parallel wires)	± 1"	± 2"
All others	± 2"	± 2"

Structures

Along track spotting tolerance		
Special trackwork locations	=	± 2'6"
At other locations	=	± 5'0"
Cross track spotting tolerance		
At restricted locations (13'–0" track spacing along semi-exclusive or exclusive ROW)	=	± 3/4"
At other locations	=	± 1½"

5.3.2 Type of Substation

General

Power supplied to LRT or HRT for the operation of trains is most commonly of direct current supplied by transformer rectifier substations. Primary power is taken from the local electric utility at high voltage AC, three phase, and transformed and rectified to a nominal DC voltage in the range of 600 Vdc to 1500 Vdc.

Direct-current traction power should be provided by traction power substations with rated voltage output at 100% load. Maximum voltage output (at rated input voltage) below 1% full load at the substation bus should not exceed design levels. Rectifier-inherent voltage regulation should be per NEMA RI-9—linear from 1% full load to 100% load and as linear as technically feasible from 100% load to 450% load.

The OCS, contact rail, running rails, and associated connections must be capable of maintaining a voltage at the vehicle no lower than vehicle design minimum operating voltage.

Train Data for Traction Power Calculations

The following constitute the basis for preliminary traction power calculations:

1. Car length
2. Empty car weight
3. Car weight with passengers (design)
4. Maximum car speed
5. Maximum rate of acceleration
6. Maximum deceleration
7. Traction motors per car
8. Vehicle voltage
9. Vehicle minimum voltage
10. Vehicle voltage at rated performance
11. Average station dwell time
12. Car auxiliaries power
13. Gear ratio
14. Wheel diameter
15. Type of motor
16. Limiting current
17. Frontal area of vehicle
18. Maximum stopping distance
19. System operation (maximum cars/train minimum headway)

5.3.3 Type of Distribution System—Overhead Contact System or Contact Rail

5.3.3.1 Overhead Contact System

The OCS includes the catenary system, the physical support system, and the associated feeder system.

The catenary system consists of the conductors, including the contact wire and supporting messenger; in-span fittings; jumpers; conductor terminations; and associated hardware located over the track and from which the vehicle draws power by means of physical contact between the pantograph and contact wire. The catenary system must provide for satisfactory current collection under all operating conditions.

The physical support system consists of foundations, poles, guys, insulators, brackets, cantilevers, and other assemblies and components required to support the catenary system in the appropriate configuration. The design must support the catenary system in accordance with allowable loading, deflection and clearance requirements. The supports throughout the system should incorporate double insulation in accordance with the requirements of regulatory agencies. Structure grounding and bonding measures must be provided in accordance with corrosion control and safety requirements.

The feeder system consists of the feeder conductors, jumpers, switches, duct-work, and associated hardware that feed the power to the catenary system. The feeder system, in combination with the catenary system, must provide for the supply of traction power to the vehicles within the allowable voltage limits.

Electrical continuity must be provided in the OCS from substation to substation. At the substations, the catenary system continuity should be sectionalized to provide isolation of each electrical section. An arrangement providing continuity and flexibility for sectionalization of the OCS while any substation is undergoing repair or maintenance should be incorporated. This can be accomplished through the application of both electrically and manually operated outdoor and indoor type disconnect switches as required for operations and maintenance.

Jumper cables should be provided to maintain electrical continuity at special trackwork locations where it is necessary to have a physical separation in the catenary system. At locations where jumper cables are used to provide electrical continuity, they should provide sufficient conductivity for the OCS circuit ampacity.

The design of the overhead contact system should be based on technical, operation and maintenance requirements, aesthetics, and economic considerations.

Subway

In a subway system, a low profile simple catenary fixed termination (SCFT) system can be used. This consists of a single contact wire and a messenger wire located over the track. The system should be fixed termination, with the result that conductor tension will vary with temperature.

The catenary should be supported by direct insulated attachment of the messenger wire to the subway ceiling, with the contact wire registered by support arms. The limited clearance requires close support spacing to minimize system depth.

The feeder system should consist of underground feeder taps from the traction power substations to the overhead contact system, including associated jumpers, switches and hardware.

Street Running

On surface streets, a single-wire fixed termination (SWFT) catenary system can be used. Cross-span or head-span construction should be used wherever practicable. This system should consist of a single contact wire located over the track.

The use of new OCS pole supports in such areas should be kept to a minimum, making use of attachments to adjacent structures and buildings, traffic light poles, and street lighting poles whenever possible in order to reduce clutter (aesthetics).

A supplementary along-track feeder system might be required to meet ampacity and power requirements. The feeders should be insulated cables located in buried cable ducts running parallel to the track.

Feeder connections to the catenary system should be provided as required, using risers at support structures.

Exclusive or Semi-Exclusive Right of Way (ROW)

On exclusive or semi-exclusive ROW, simple catenary auto-tension (SCAT) system should be used. The catenary system should consist of a messenger wire with a single contact wire supported by vertical hangers. The system should be designed to meet ampacity and power requirements without the use of supplementary along-track feeders. The system can be auto-tension by means of weight-tensioning devices located at the termination points of the conductors. Tension in the conductors, should remain constant up to the conductor temperature of 130°F, after which a resulting increase in temperature is accompanied by a decrease in tension up to a maximum temperature of 165°F. The physical support system should consist of concrete foundations, tapered tubular steel poles, and hinged cantilevers. Center poles (between tracks) with back-to-back cantilevers should be used wherever practicable.

The feeder system should consist of underground feeder taps from the traction power substations to the overhead contact system, including associated jumpers, switches, and hardware.

Main Yard and Shops and/or Maintenance-of-Way Satellite Yard

In the yards and shops, a single wire fixed termination (SWFT) system can be used. This consists of a single contact wire located over the track. The terminations of the contact wire should be made directly to the poles with the result that conductor tension will vary with temperature.

The poles should be tapered tubular steel, or any other type that is designed to be consistent with the rest of the system. Wherever practicable attachments to the exterior walls of the shops should be used. The contact wire should be supported by cross-span wires and cantilevers. On curved track, backbone systems can be used to minimize the number of poles.

A feeder system is required to meet power requirements. The feeders should be insulated cables located in buried cable ducts wherever practical and routed in accordance with the yard/shop sectionalizing scheme. Feeder connections to the catenary need be provided as required, using risers at support structures. Switches and associated hardware should be provided for sectionalizing.

Operations

The OCS must be designed for vehicle operations as determined. A design margin of ten miles per hour (10 mph) should be allowed over the specified maximum vehicle operating speeds.

The OCS may need to be designed for multiple pantograph operation with pantographs spaced in accordance with the specified train consists.

For purposes of the OCS design, it should be assumed that vehicles will not operate on the system when wind speeds are in excess of 55 mph.

Sectionalization

The OCS catenary should be electrically sectionalized, consistent with the location of the traction power substations, the track layout, the signaling scheme and proposed operations. The sectionalization should allow sections of the OCS to be de-energized for maintenance and emergency purposes.

On the exclusive or semi-exclusive ROW where the SCAT system can be used, the OCS catenary should be sectionalized by means of insulated overlaps wherever possible. In the event that an insulated overlap is not possible, mechanical section insulators should be used.

In cases of subways, where low-profile SCFT catenary can be used, the OCS should be sectionalized by means of insulated overlaps. Mechanical section insulators should be used at crossovers.

In areas where the SWFT system is used, the OCS should be sectionalized by means of mechanical section insulators.

Inside shop buildings, the OCS catenary should be sectionalized at each entrance location to the building and at other locations as determined.

Span Lengths and Staggers

The span lengths (spacing between contact wire registration points) and staggers should be designed to provide for pantograph security (i.e., no pantograph de-wirement) and to maintain good current collection and uniform wear of the pantograph carbon collector. Pantograph security is established by maintaining a minimum contact wire edge distance (from the tip of the pantograph) of 6 inches (3 inches at overlaps) under worst operating condition. In addition, the contact wire must be staggered to provide for uniform pantograph wear.

The design should consider the effects of environment, track geometry, vehicle and pantograph sway, and installation and maintenance tolerances. Vehicle roll into the wind should be taken equal to 50% of the maximum dynamic roll value in accordance with AREA Manual, Committee recommendation, Bulletin 694.

The determination of span lengths for single wire systems must take into consideration the requirements of State and/or local regulations regarding broken OCS suspensions and fastenings.

Catenary Conductors

The contact wire should be solid grooved hard-drawn copper conforming to ASTM Specification B47.

The messenger wire should be standard hard-drawn copper conforming to ASTM Specification B189 with stranding conforming to ASTM Specification B8, class B or higher.

Conductor tensions must be in accordance with the requirements of State and/or local regulation. Thirty percent cross-sectional area loss due to wear of the contact wire and the effect of temperature change should be taken into consideration in the design for conductor tension.

Feeder Conductors

Feeder conductors should be insulated, non-shielded, single conductors suitable for use in wet or dry locations and rated at least 2,000 VDC, or at voltage consistent with system design voltage, 90° conductor temperature for normal operation, 130°C for emergency operation, and 250°C for short-circuit conditions. The conductors may be copper, conforming to ASTM B189 material with Class C stranding, and conforming to ASTM B8 with EPR insulation and low smoke jacket.

Traction power cables connecting DC feeder breakers to the overhead contact system and from running rails to the negative bus must be sized to accept maximum overload and short-circuit currents with a temperature rise not to exceed safe insulation design limits of the cables.

Feeders should be standardized on a single-conductor size by using multiple conductors for different ampacities. The feeder cables must have sufficient conductivity to maintain traction power voltage levels at the LRV and current ampacities as determined. The traction power feeder cables must be sized to operate at rated cable insulation temperature during normal operating conditions.

At points of entry and exit of underground feeders, protection against current surge due to lightning strokes should be provided.

Feeder Ductwork

Feeder ductwork can be buried underground and should consist of polyvinyl chloride (PVC) or fire retardant FRE conduit encased in concrete. Design of ductwork such as conduit size, design cable pull, maximum total angular turn, minimum embedment depth below grade, manhole spacing and duct gradient should be in accordance with NEC requirements. Feeder ductwork should be identified by a yellow warning tape 6 inches wide marked "Warning—High Voltage," laid 12 inches above concrete encasement in backfill.

Feeder ductwork should be run as directly as practicable and should be located to avoid interference with foundations, piping and other similar underground work. Risers consisting of PVC coated galvanized rigid steel conduit can be provided at feeder connections to the catenary system.

Contact Wire Heights and Gradients

Minimum contact wire heights must be in accordance with the requirements of the local regulatory agencies. At railroad crossings, contact wire height should be a minimum of 24' or in compliance with State and/or local regulations. Maximum contact wire gradients should be in accordance with the AREA Manual, Chapter 33, Part 4.

The contact wire height at supports should take into consideration the effect of wire sag and installation tolerance (including track construction and maintenance tolerances).

Structure Design

OCS support structures must be designed to carry the design loads according to the requirements of allowable stresses and deflections.

Local regulations should be used except where more stringent AISC and ACI requirements for steel and concrete design are applied.

Design Loads

OCS support structure design loads must be the system self-weight plus the loads indicated in State and/or local regulations, Light or Heavy Loading.

Self-weight must be the actual weights of poles, cantilevers, assemblies and conductors computed according to the AISC Manual of Steel Construction or obtained from manufacturer's catalogs, as applicable.

Wind loads must be determined in accordance with State and/or local regulations.

The design load should be multiplied by the following overload factors to allow for uncertainties in loading conditions.

Design for strength = 1.1
Design for deflection = 1.0

Design For Strength

Steel poles, cantilevers and other structures should be designed by the allowable stress method according to the AISC Specification for the Design, Fabrication and Erection of structural steel for Buildings.

Reinforced concrete drilled pier foundations can be designed by the ultimate strength method according to the local Building Code Requirements for Reinforced Concrete; anchor bolts should be designed by the alternate method (working stress method). The anchor bolts should be designed based on ungrouted pole baseplate.

Laterally loaded pier foundations can be proportioned according to the Texas Transportation Institute, Resistance of a Drilled Shaft Footing to Overturning Loads—(Research Reports 105-1, 2 and 3). A minimum factor of safety (to failure of soil) of 2.0 should be used in the design.

For combined dead plus live (wind) loading, the 33 percent increase in allowable stress can be waived.

Design for Deflection

OCS support structures must be designed so that structure deflections under service loads will not cause excessive movement of the contact wire. In addition, the steel pole can be raked to compensate for the deflection generated by the self-weight and conductor tension loading.

Design of support structures should be based on the following criteria for deflection and foundation rotation:

Structure	Loading	Maximum Deflection	Remarks
Steel Pole	Live (wind)	2½" (at contact wire level, excluding foundation rotation effect).	Total deflection at contact wire level including foundation rotation effect shall be ≤ 4".
	Dead + Live	2.5% of pole height	
Foundation	Live (wind)	0.5% rotation	
	Dead + Live	5.0% rotation	

Seismic Design

OCS support structures should be designed to conform to the seismic design requirements where applicable.

OCS Grounding and Bonding

The OCS must be grounded in accordance with NEC requirements. Generally, footing resistance of individual structures should be maintained at a maximum of 25 ohms. If necessary, ground rods should be installed.

Ground connections to disconnect switches and ground leads on all surge arresters should have a maximum ground resistance of 5 ohms. Ground rods can be utilized to obtain the required ground resistance. OCS support poles must be bonded to the concrete pier foundation.

5.3.3.2 CONTACT RAIL

Contact rail can be of two general types:

1. Overrunning
2. Underrunning

In the overrunning type the rail is of low-carbon steel or steel rail with aluminum cladding huck-bolted to the web for added current carrying capacity at minimal weight. 80 lb. steel rail with aluminum cladding can increase current carrying capacity from 800,000 CM to 4.6 MCM. The rail can be supported on porcelain or fiberglass insulators which are supported on extended ties (approximately every sixth tie) or by direct fixation.

With the underrunning type contact rails, the collector device on the vehicle maintains contact with the contact rail on its lower surface. The contact rail can be supported by brackets or other suitable supports.

Generally contact rail in the United States for heavy rail transit is of the overrunning type as described below:

The contact rail for mainline tracks should have an electrical resistance not greater than 0.002 ohms per thousand feet at 20°C and should be capable of carrying at least 4,000 Amp DC or more, according to vehicle design and operation, continuously at a temperature rise not exceeding 40°C over a 30°C ambient in still air. Contact rail for a yard area should have an electrical resistance not greater than 0.004 ohms per thousand feet at 20°C, and should be capable of carrying at least 2,000 Amp DC or more, according to vehicle design and system operation, or special yard loading such as vehicle pre-heating or cooling, continuously at a temperature not exceeding 40°C over a 30°C ambient in still air.

Contact rail height should allow sliding of current collector shoes on top of contact rail when the contact rail is seated upon support insulators with at least eight inch (8") leakage path to the supporting tie. The top wearing surface of the contact rail should be at least two inches (2") wide to lessen wear.

The support insulator should be centered below the contact rail and the insulator base should be sufficiently wide to provide a stable arrangement for the rail. The contact rail and support insulator should withstand, without permanent deformation, the stresses caused by the maximum short circuit forces.

Contact rail joints should not have misalignment or roughness. Bolted butt joints should have minimum gap between rail ends and be ground smooth for maximum wear and abrasion of collector shoes.

Feeder connections to contact rail should be designed, located, and attached to provide permanent connection without excessive protrusion from the side of the rail. The standard rail lengths should be connected by means of bolted or welded joints.

The relative position of the contact rail to the running rails should be coordinated with the design of the vehicle current collector.

The standard contact rail lengths should be not less than 39 feet nor more than 60 feet plus or minus one percent. The rail should have sufficient section modules so that the maximum deflection with a concentrated load of 25 pounds at midpoint between support insulators placed 10 feet apart should be not more than 1/64 inch.

Protective cover should consist of a curved insulating board covering the top of the contact rail. Side cover-board should be provided where the contact rail is adjacent to safety or maintenance walkways. Protective coverboards should have adequate clearance to ensure no obstruction of current collector shoes movement and to permit insertion of shoe paddles.

The protective cover-board should extend a minimum of 12 inches beyond the tip of the end approach.

Contact rail through stations should be located at trackside opposite the platform.

Contact rail at grade should be located in the area between running tracks, except at yard areas, special trackwork, and through center platform stations.

In sections of contact rail of 2,000 feet or less, a contact rail anchor should be provided at midpoint; otherwise, rail anchors should be provided at maximum 1,000 foot intervals at mid-point between expansion joints. Spacing of anchors should be adjusted to provide an anchor near the middle of curved sections, with expansion joints at point of tangent.

The contact rail should be physically continuous between substations except at crosswalks and special trackwork locations or sectionalizing points where it is necessary to have separations in the contact rail. End approaches should be provided at each separation to facilitate vehicle current collector shoe return to the contact rail without significant bounce.

The design of the entire contact rail system should ensure that, during normal operation, at least one current collector shoe of a two-car train is always in contact with the rail.

The contact rail system should be electrically continuous. At cross walks or special trackwork locations (or around expansion or sectionalizing joints), electrical continuity should be provided by jumper cables either bolted or welded to the contact rail. At substations, electrical continuity should be provided via dc switchgear, and at wayside locations via motorized disconnect switches connected to the contact rail by cables. The disconnect switches and cables should provide conductivity that will not reduce the circuit capacity of the contact rail.

Contact rail sectionalizing at substations and at the locations of the wayside disconnect switches (as required to provide definite traction power zones) should be implemented by means of non-bridgeable type gaps. The length of the non-bridgeable gap between power zones should be such that it cannot be bridged by front and rear shoes of a transit vehicle. In the vicinity of passenger stations, each non-bridgeable gap should be located preferably in the normal decelerating zone. At stations where it is more economical to locate gap locations other than at stations, the gap should be of special design to prevent interruption of power to trains during normal operation.

DC Power Cables

A. All DC traction power cables should be stranded, nonshielded copper conductors.

B. The traction power cables connecting the DC feeder breakers or wayside dc disconnect switches to the contact rail, and from the running rails to the negative bus should be sized to accept maximum overload currents and a temperature rise not to exceed safe insulation design limits of the cables, based on a minimum insulation life of 40 years.

C. The cables should have sufficient conductivity to maintain traction power voltage levels within the limits determined, confining the major voltage drop to contact and running rails, rather than permitting excessive voltage drop in the connecting cables.

D. Traction power feeders for each power zone should have cable ampacity as required, by the ratings of the associated dc feeder circuit breakers. These ampacities should not be compromised by virtue of different types of raceway arrangements for various sections of the feeder.

E. Negative cable should be provided between the substation negative bus and the connection to each pair of the running rails. Current carrying capacity of negative cable per each pair of running rails should be equal the current carrying capacity of positive cable per each DC feeder circuit.

F. Since the contact rail constitutes a vibrating mass, provision should be included in the design of all cable terminations to the rail to assure no cable failures. The design should utilize standard stranding feeder cables terminating at a junction box adjacent to the contact rail, with extra-flexible stranding cables provided for the final bolted or welded connection to the rail.

G. Feeders should be of a common conductor size, using multiple conductors for the different ampacities. Conductor size should be selected so as to minimize installation costs.

5.3.4 High Voltage Sub-Transmission Lines

Substation Power Supply

The local utility provides 3-phase, 60 Hz power circuits as primary service.

The alternative primary service voltages and methods of primary power distribution to the traction power substation should be evaluated. The evaluation can determine the most cost-effective investment and lowest annual operating cost that will provide adequate and reliable service to the LRT or HRT system.

High-Voltage AC Power Cables

The high-voltage power cables for interconnection of substation equipment should conform to the following requirements:

1. All cables for incoming service to a traction power substation are usually furnished and installed by the serving utility up to agreed-upon interface point, and in accordance with their standards.

2. Minimum cable size must be based on the system maximum available short-circuit currents.

3. Cable ratings must be based on a 100% demand load factor and the 2-hour (overload) rating of the rectifier equipment. Cable size should be based upon the circuit breaker fault clearing rating of 8 cycles and a temperature rating of 90°C for normal operation, 130°C emergency overload, and 250°C for short circuit.

4. Sizes of AC high-voltage cables installed in air or in conduit in air should be based on the current carrying ratings of one circuit in air temperature with a spacing between loaded cables that is adequate to prevent mutual heating.

5. Sizes of high-voltage AC cables installed underground should be based on earth temperature with appropriate derating factors applied for the number of loaded circuits in the underground duct bank.

6. AC high-voltage cables in conduit or underground ducts should be single conductor shielded type preferably with copper conductors.

7. AC high-voltage cable installed on racks should be three-conductor shielded type preferably with copper conductors.

8. All AC high-voltage power cables should be moisture-proof and should pass the IEEE 383 flame test.

5.3.5 Protection and Remote Control/Monitoring

Substation Equipment (Light Rail Vehicle)

Substations can be prefabricated units or individual components equipped with high voltage AC switchgear, transformer-rectifier units, DC power switchgear and can be designed to operate unattended. Controls can be provided to operate all switchgear from the control center or from local control switches in the equipment. Transformer-rectifier units should be connected in accordance with ANSI Standard C34.2; mainline and yard units can deliver a 12-phase, double-way output, and the shops unit can deliver a 6-phase (or 12-phase if harmonic problems exist), double-way output. Rectifier transformers should be self-ventilated dry-type Class AA, suitable for indoor service. Silicon diode rectifiers should be free-standing indoor type metal enclosed, natural convection air-cooled. The transformer-rectifier units should be rated in accordance with NEMA RI-9; mainline units should be extra-heavy traction rating class, and shops unit should be medium traction rating class.

The cathode and feeder breakers should be indoor, metal enclosed, draw-out, single-pole, semi-high speed capable of interrupting the maximum short-circuit current available. Auxiliary equipment should include lightning or surge protection; interconnecting buswork; control-power battery and charger; provision for stray-current control drain cables; and provision for interfacing with the Supervisory Control And Data Acquisition System (SCADA).

Circuit breakers should be installed to provide isolation of track sections.

Circuit breakers should be equipped with direct acting instantaneous over-current "rate-of-rise" and automatic reclosure relaying. At the substation, circuit breakers should be used for all feeders.

Circuit breakers can be operated remotely from the control center by means of the Supervisory Control or SCADA System. Local controls must be provided to permit local manual operation of circuit breakers within each substation. During local operation, supervisory control for each device operated locally should be bypassed. Substation interconnecting buses can be copper with silver-plated joints at bolted connections. Buses must be sized on the basis of NEMA standards and must be adequately supported to withstand available short-circuit current at the appropriate bus-voltage level.

The primary service cables from the utility company should be extended to the substation incoming-line cubicles by the power company. Required duct banks may be designed by the utility company and can be installed by the traction power contractor. Power company metering provisions must comply with the company's requirements. Where incoming service from overhead lines is subject to lightning surges, adequate protective lightning arresters should be provided.

Substation Equipment (Heavy Rail Vehicle)

A. Substations should be equipped with unitized transformer-rectifier assemblies. Protective relaying should be compatible with the requirements of serving utility to provide selective tripping.

B. Substations should be designed to operate unattended. Controls should be provided to operate all switchgear both from Operation Control Center (OCC) or from local control switches in the substation.

C. High voltage AC switchgear must be rated in accordance with ANSI Standard C37.06.

D. Transformer-rectifier units should each be rated in accordance with NEMA Standards Pub. No. RI 9-1968 (R1979) for extra-heavy traction service with the possible exception for main shop unit (if used) which can be rated for medium service. Output voltage of transformer-rectifier units shall be of nominal system design voltage at 100 percent rectifier load. Inherent voltage regulation of transformer-rectifier units should be a maximum of 6 percent between 1 percent and 100 percent full load.

E. Rectifier transformers should be dry-type indoor, and dry-type or liquid -immersed-type outdoor, and can be rated up to 34.5 KV depending on the serving utility.

F. Silicon diode rectifiers can be indoor type, air cooled, and self ventilated. A redundant diode in each parallel group should be included. Each rectifier unit should be connected for 12-phase operation with the exception for main shop rectifier (if used) which can be connected for 6-phase operation if harmonic levels permit.

G. The DC main and feeder circuit-breakers should be single pole, high speed, or semi-high speed, as required.

H. Auxiliary equipment should include interconnecting buswork where practical, 125 volts DC control power supply, provision for future stray current corrosion control drain cable, and interface with the Supervisory Control and Data Acquisition (SCADA) system.

I. All interconnecting buses should be either tin-plated aluminum, tin-plated copper, or copper with silver-plated joints at pressure connections. Buses should be sized on the basis of applicable standards. All buses should be adequately supported to withstand available short circuit current stresses at the appropriate bus-voltage level.

J. The primary service cables from the utility company must be extended into the space allocated for the utility company service equipment. Two imbedded PVC or ABS conduits should be provided for the utility company primary service cables. Utility company service equipment should be installed as required by their standards, and in accordance with agreements with them.

K. Underground service should be recommended wherever feasible; however, where incoming service is from overhead lines subject to lightning surges, adequate protective lightning arrestors should be provided.

Equipment Arrangement

Substation housing should have adequate area to accommodate traction equipment and ancillary components. The arrangements of the equipment must permit doors to be opened, panels to be removed, and switchgear removable elements to be withdrawn. Ceiling heights and structural openings must permit entry and removal of the largest components installed in the structure.

If substations are located below grade they should be constructed with equipment hatches to permit the removal of equipment from the substation to the ground level above or doors to permit the removal of equipment from the substation to track level for loading onto flat cars.

Where cable trenches, under-floor raceways, pits, or embedded conduits are provided beneath switchgear, the trenches, pits, or conduits should be initially extended to permit addition of future equipment, if planned. All pits or trenches so provided should be furnished with safety covers for use until future equipment is installed. Conduit setups should be capped. Adequate drainage should be provided wherever water may be present.

Lighting

Indoor lighting can be provided by fluorescent fixtures. Design should provide for minimum maintained lighting levels of 30 foot candles vertical, average. Such lighting should be located so as to illuminate satisfactorily the vertical surfaces of equipment such as switchgear and transformer rectifier units. Locations of lighting fixtures should be coordinated to avoid interference with overhead raceways or other major wiring and should not be directly above switchgear, rectifiers, or transformers. Outdoor lighting can be provided by sodium lamp fixtures with unit photo cell control. Design should provide a minimum illumination level of one foot candle at ground level. The general lighting should be controlled from switches located near each access door.

Emergency Lighting

Substations should be provided with emergency lighting from individual unit equipment consisting of rechargeable lead acid batteries and battery chargers, with one or more lamps mounted on the equipment and a relaying device arranged to energize the lamps automatically on failure of the ac power. The battery should have the capacity to supply rated load for 1.5 hours at not less than 87.5% nominal battery voltage.

Sufficient fixtures should be provided to illuminate egress paths as required by code(s).

Convenience Outlets

Duplex convenience outlets should be located approximately 25 feet apart around the interior walls of the substation. One 20 A duplex outlet near the switchgear and rectifier should be separately circuited to permit use of a heavy-duty vacuum cleaner or up to 1/2-horsepower portable air compressor.

Feeder Supports

Traction power positive cables from the DC feeder breaker connections and negative cables from the negative bus connections should be run in appropriate raceways such as metal or nonmetallic trays, nonmetallic conduit, cable trenches, or on racks through the substation. Raceways should provide adequate cross-sectional area to permit a neat alignment of the cables and to avoid crossing or twisting (where laid).

On racks, porcelain or fiber cable-support insulators designed for this purpose should be used. Such supporting racks should be spaced to avoid excessive weight or pressures on the cable insulation. The cables should be arranged in not more than two layers. Positive and negative cables must be run in separate raceways.

Design of supports shall be in accordance with the requirements of the NEC. The design should be coordinated with the signaling and communication cabling design as well as with that of the other traction facilities. In particular, feeders should be located such that possible interference, both mechanical and electromagnetic, should be minimized.

5.3.6 Negative Return Circuit

Rail Bonding

The rails should be welded in continuous lengths and bolted joints must be electrically bonded. At locations requiring insulated joints, the traction power direct current continuity of negative rails must be maintained by use of impedance bonds.

In areas of double track equipped with vital double-rail AC track circuits, cross-bonding between tracks for negative-traction, current-return equalization should be accomplished by impedance-bond center-tap connections at each substation return feeder location. In no instance should consecutive impedance bonds be used for either cross-bonding or substation return connections. In areas of trackage equipped with single-rail AC track circuits, cross-bonding between tracks should be accomplished by direct connections between the negative traction return rails only. In areas of trackage not equipped with track circuits, cross-bonding between tracks should be accomplished by direct connections to both running rails.

Impedance bonds should have a minimum ampacity of 1,000 amps continuous "on" with short time rating of 5000 amps per rail. Impedance bond center tap connecting cables and cross-bonding cables should be 750 kcmil insulated copper, the number should be determined by load requirements at each location. Track ballast needs to be clean, dry, and well drained, and should not contact the running rails for mitigation of stray current and loss of shunting or calibration with signal systems.

5.3.7 Grounding and Bonding

Substation Grounding

Each substation should be equipped with a copper ground bus and necessary cabling to a substation grounding grid.

Noncurrent-carrying metal enclosures or parts of alternating current equipment, including ac apparatus and rectifier-transformers, should be securely connected to the ground grid.

Enclosures for traction power rectifiers, DC switchgear, and DC busways should be installed insulated from ground on their support surfaces, and each should be connected to the substation ground grid through a ground fault detection system.

Where metallic structures interconnect grounded and ungrounded equipment, e.g., the bus connection between rectifier and rectifier transformer, adequate insulated sections should be provided. The main negative bus of each traction power substation should be connected to the return negative bus through a shunt for current measurement purposes.

The DC systems are normally operated ungrounded. An insulated floor surface (epoxy, rubber, neoprene, nylon, or other insulating material) should be provided around and under all ungrounded enclosures and should extend five feet (5') beyond the enclosure on all sides.

Traction power rectifier transformer output (DC) windings must be isolated from ground.

The ground grids can consist of driven ground rods and conductor mats embedded in the earth, and should be designed for safe step-and-touch potentials. These grid materials should be resistant to corrosion by the earth's chemistry. (See 5.5.1 Corrosion Control and Protection)

5.3.8 Stray Current Control (also see 5.5.1)

To minimize stray currents and to provide a means of monitoring such currents in the affected structures and utilities in the proximity of LRT facilities, the following provisions should be considered:

- The mainline traction power system can be isolated from the yard(s) and subway segment.
- Rails should be insulated from direct contact with ground by means of insulated pads.

- Cross bonds should be installed between rails at appropriate locations.
- Test stations can be established to facilitate measurements.
- Electrical continuity between pole structure and foundation should be maintained.

The mitigation measures listed above should be coordinated with corrosion control consultants.

5.3.9 Electrical Characteristics of Running Rail

A. The running rails are normally 115 pounds per yard, per Section AREA specifications, 1962.

B. Both running rails of each track should serve as negative conductors except at special trackwork or other special rail conditions. Rail should be welded in continuous lengths. At locations requiring insulated joints, the traction power DC continuity of running rails should be maintained by use of impedance bonds.

C. Running rails should be cross-bonded for traction power equalization through impedance bonds at every traction power substation where ATO is used, as a minimum. Intervals between consecutive cross-bonds should not exceed 1.5 miles. The locations of impedance bonds used for cross-bonding should be coordinated with the design of the signal system or Automatic Train Control System.

D. Running rails should be insulated from roadbed and insulated track fasteners should be used. Track ballast should be clean, dry, and well-drained, and should not contact the running rails.

5.3.10 Harmonics

Harmonics are generated by the transformer-rectifier units and should eliminate effectively any harmonic currents from being generated. 12-phase operation and alternate “delta-wye” windings of adjacent transformers where multiple transformer-rectifier units are housed in the same substation and the use of filters are some of the methods to eliminate unwanted harmonics.

5.4 TRACTION POWER EQUIPMENT (TO BE FURTHER DEVELOPED)

5.4.11 Supervisory (Central) Control Interface

Supervisory Control Indications

All the essential functions of switchgear operation and control can be performed at Control Center by means of a Supervisory Control And Data Acquisition (SCADA) system. All essential indications of instruments and meters at the traction power substations (such as voltage level, circuit breaker positions) can be telemetered for selective display at Control Center.

A communication interface cabinet (CIC) or (PLC) should provide the hardware interface between the substation equipment and the SCADA system. Substation equipment should also be provided with suitable control devices to permit local operation. During local operation of any equipment, remote supervisory control functions for that equipment should be disabled by means of a “LOCAL-REMOTE” control selector switch.

The control operation of a typical traction power substation is described below.

A. Substation Start-Up Sequence

The following depicts the start-up sequence for a typical rectifier substation from Operation Control Center:

1. Select substation
2. Select and close incoming line high-voltage AC circuit breaker
3. Select and close rectifier transformer high voltage AC feeder circuit breaker or load interrupt switch

4. Select and close rectifier main DC circuit breaker
5. Select and close DC feeder circuit breaker

B. Conditions Preventing Start

The conditions which will prevent start-up of a rectifier substation remotely should include but not be limited to:

1. Incoming line high-voltage AC circuit breaker, lockout trip relay in operated position
2. Equipment LOCAL-REMOTE switch in LOCAL position
3. Loss of control voltage
4. Transformer-rectifier unit, lockout trip relay in operated position

C. Substation Shutdown Sequence

The following depicts the shutdown sequence for a typical rectifier substation from Operation Control Center:

1. Select substation
2. Select and trip DC feeder breaker
3. Select and trip rectifier main DC circuit breaker
4. Select and trip rectifier transformer, high-voltage AC feeder circuit breaker or load interrupter switch
5. Select and trip incoming line high-voltage ac circuit breaker

D. Transformer-Rectifier Lockout

The following devices should operate the transformer-rectifier lockout relay and cause shutdown until it is reset by hand in the substation. One continuous alarm signal should be transmitted to Operation Control Center to indicate a transformer-rectifier lockout. The specific device that caused the lockout should be indicated on the local annunciator panel in the substation, including but not limited to:

1. Transformer winding over-temperature device, second step
2. Transformer sudden pressure device (liquid immersed only)
3. Rectifier over-temperature device, second step
4. Rectifier hot-structure ground relay (if used)

E. Operation Control Center Supervision

A power supervisory console should be provided at the Operation Control Center for the supervision of substation and wayside electrical apparatus. In addition, changes in the status of substation and wayside electrical apparatus should be placed in a direct access memory storage system with the time and date of entry for diagnostic and documentation purposes. The normally unattended substations should be continuously supervised from Operation Control Center including but not limited to the following:

1. Substation high-voltage AC rectifier
2. Rectifier main dc circuit breakers
3. Substation secondary feeder circuit breakers (AC & DC)
4. Wayside disconnect switches
5. Emergency trip stations

6. Substation equipment alarms
7. Substation analog metering including DC voltage

A partial list of substation and OCS supervisory control points is suggested in Table A below:

Table A. Traction Power Supervisory Control Points

	Remote Functions		Local Functions
Equipment	**Control**	**Indication/Alarm**	**Indication/Alarm**
AC SWITCHGEAR			
Main Incoming Breaker	x	x	x
O/C Trip		x	x
Motorized Fused Load Break Disconnect Switch	x	x	x
AC-DC CONVERSION EQUIPMENT			
*Transformer Winding Over-temperature		x	x
*Rectifier Over-temperature		x	x
*Rectifier Diode Failure		x	x
*Rectifier AC Surge Suppressor		x	x
*Rectifier DC Surge Suppressor		x	x
Rectifier Enclosure Alive		x	x
Rectifier Enclosure Grounded		x	x
DC SWITCHGEAR			
Cathode Breaker	x	x	x
Feeder Breakers	x	x	x
Pilot Wire Transfer Trip Failure		x	
Reclosure Failure		x	
DC Switchgear Enclosure Alive		x	x
DC Switchgear Enclosure Grounded		x	x
OCS De-energized		x	x
MISCELLANEOUS			
Plugged Air Filter		x	
Intrusion Detection		x	
Fire Detection			
System Trouble		x	
System Power Supply		x	
Fire Alarm		x	
Loss of Control Voltage AC Switchgear		x	x
Loss of Control Voltage AC-DC Conversion Equipment		x	x
Loss of Control Voltage DC Switchgear		x	x
Loss of Control Voltage Supervisory Cabinet		x	x
Battery Grounded		x	x
DC Bus Voltage		x	x
Emergency Trip Switch @ Substation Building		x	

*Remote Indication Alarm/Functions listed above should be combined into a single Indication/Alarm point by equipment designation; alarm should annunciate "AC-DC conversion equipment trouble."

5.4.12 UPS

Equipment for which a power interruption of greater than 1/4 cycle duration may cause a malfunction should be classified as critical loads requiring a UPS.

A UPS System might be required by State and/or local regulations in subways or similar environments to provide battery powered emergency lighting for the evacuation of people and/or to keep safety sensitive systems operable to accomplish evacuation.

Signalling or ATO are systems where battery back-up or UPS might be required.

5.4.13 ETS System (see 5.2.5.6) (to be further developed)

5.5 SPECIAL CONSIDERATIONS

5.5.1 Corrosion Control and Protection

Three types of corrosion control need to be addressed and protected against:

1. Stray Current Corrosion

   A. Stray earth current generated by normal system operations should not exceed 0.20 ampere per 1,000 feet of system.

   B. Positive or negative traction power distribution circuits should not have direct or indirect electrical connections to earth.

   C. Ancillary systems and equipment connected to either the positive or negative traction power distribution circuits should not contribute more than 5% of the system earth conductance.

   D. Water infiltration into the trackway area should not contact the rails, fasteners and/or conductive rail appurtenances during normal system operations.

2. Soil Corrosion (corrosion caused by soils and groundwater)

   A. Pressure and nonpressure piping and conduit should be non-metallic, unless required for specific engineering purposes. Aluminum and aluminum alloys should not be used for direct burial purposes. Where this is not practical, buried metallic pressure piping may require cathodic protection, and buried nonpressure piping may require corrosion protection.

   B. Corrosion control may be required for those facilities, where failure caused by corrosion may affect the safety of or interrupt continuity of operations.

   C. Six inches should be provided between new and existing metallic structures. When conditions do not allow a six inch clearance, electrical contact with structures needs to be protected against.

   D. Electrical continuity should be provided for all buried non-welded pipe joints.

3. Atmospheric Corrosion

Criteria for atmospheric corrosion should be developed for the preservation of structure appearance and reduction of maintenance costs.

Corrosion control engineering should be interfaced and coordinated with other disciplines, including, mechanical, utility, electrical, civil, structural, trackwork, electrification, signaling, and communications designs.

(FOLLOWING SECTIONS TO BE DEVELOPED)

5.5.2 Compatibility with Train Control

5.5.3 Street Level Running Considerations (LRT)

5.5.4 Elevated Construction

5.5.5 Electromagnetic Interference and Compatibility

5.6 TESTING AND ANALYSIS

5.6.1 Safety Certification Considerations

Codes and Standards

In general, the Rail Transit Electrification System design must be in compliance with and certified by local regulatory agencies.

Additional codes and standards should be applicable to specific aspects of the design.

In all cases, latest editions of the codes and standards must be applicable.

All materials, apparatus and equipment, installing methods, and testing should conform to or exceed the requirements of the latest ANSI, NEMA, NEC, IEEE, UL, ICEA, EIA, ASTM, AREA, and other standards, as applicable.

(TO BE DETERMINED)

5.6.2 System Integration and Interface Management

5.6.6 Maintainability and Reliability

5.6.7 Start-up and Testing

5.6.8 Safety

Proposed 1996 Manual Revisions to Chapter 14—Yards and Terminals

Part 4—Specialized Freight Terminal

Page 14-4-7. Add (See Section 4.3.5) at the end of the following sub-paragraphs.

4.3.1.1 General, sub-paragraph 3. Security

4.3.1.1 General, sub-paragraph 4. Lighting

Page 14-4-11. Delete all of the following sub-paragraphs

4.3.1.2 Design Considerations, sub-paragraph 5. Security

4.3.1.2 Design Considerations, sub-paragraph 6. Lighting

Page 14-4-13. Add all of new section 4.3.5 Security

4.3.5 Security

4.3.5.1 Introduction

Rail served auto terminals are specialized facilities designed to transfer autos, trucks, and other vehicles to and from rail cars. Their designs are as unique or individualized as the companies that construct and operate them. The design criterion, however unique, has a common denominator, *security*. Security not only protects the customer's commodity, but provides a safe working environment for all employees. Security can be enhanced through various methods, including lighting, fencing, barriers, gates, alarms, closed circuit television, card access systems, signs, security guards, or through any combination of these methods.

4.3.5.2 General

The level of security commitment can be a direct result of facility design or operational concept. It is also influenced by citing environmental demands, local building codes, capital commitment, volume of traffic, history of thefts or vandalism in area, and combined day/night operation.

4.3.5.3 Influence of Operational Concept

Currently there are two major methods utilized by the trucking companies that pick up and deliver vehicles to the facility; 1) standard or end loading (Figure 1), and 2) perimeter loading (Figure 2).

Trucks that use end loading never actually enter the vehicle baying or rail car areas. They back their truck up to a fixed barrier, which should be part of the perimeter barrier, drop their ramps over the barrier, and load or unload vehicles onto or from their trucks. Fixed ramps are also utilized in the same manner. This method ensures that the integrity of the vehicle storage area is maintained. No trucks are permitted in the vehicle baying area.

Although originally end loading was the standard for most auto facilities, this method is rapidly giving way to perimeter loading due to perceived operational efficiencies of the latter. Trucks using the perimeter system actually enter the vehicle storage area, and as a result, security demands are increased due to the required monitoring of the additional vehicles and personnel in the storage area. This monitoring may require security guards and/or electronic card reader systems. Exit and entry gate design, as well as camera systems, are influenced by this additional liability.

4.3.5.4 Physical Design Criteria

a) *Lighting*

Proper lighting provides a safe working environment for employees and customers. It helps prevent theft and vandalism of a shipper's product by enhancing the power of the human or electronic observer. It can also act as an effective psychological deterrent.

At this time, high pressure sodium lighting has proven to be the most efficient and cost effective in security applications. It provides more than twice the illumination of a standard mercury vapor light. Depending on the size and shape of the facility, 200, 400, and 1000 watt high pressure sodium lights should be considered. Every effort should be made to maintain a minimum of 1 foot candles throughout the facility, with an average of 1.5 foot candles.

Additional localized lighting will be required for facilities with camera monitoring or where truck loading, or other operations, is prevalent at night.

All light poles should be located as far from the perimeter fence as possible .

b) *Perimeter Barriers*

Perimeter barriers prevent the unauthorized removal of vehicles from the facility. The barrier should be within or a part of the perimeter fencing and completely encompass the interior except those areas protected by gates.

Barrier types include scrap rail, standard highway barriers, pipe, horizontal rails in fences, bollards cable, and concrete.

Barriers should be of a sufficient strength and planted to a depth as to withstand a direct impact by a vehicle.

In facilities using an end loading or standard concept, barriers in the loading/unloading area should be just low enough to allow truck ramps to clear.

c) *Fencing (or Walls)*

Proper fencing can prevent the unauthorized entry of persons onto a facility. A fence or wall should completely surround the facility with exit/entry gates incorporated into the system.

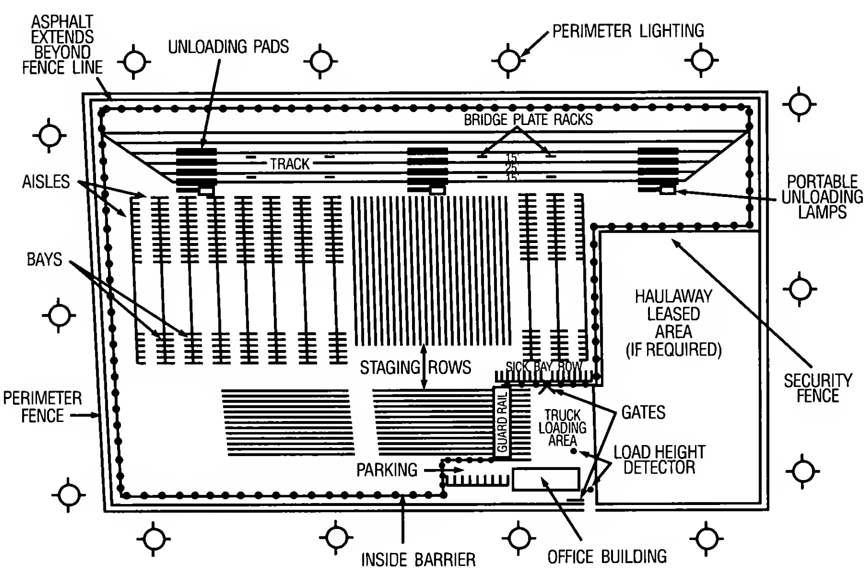


Figure 1. Suggested Automotive Handling Facility.

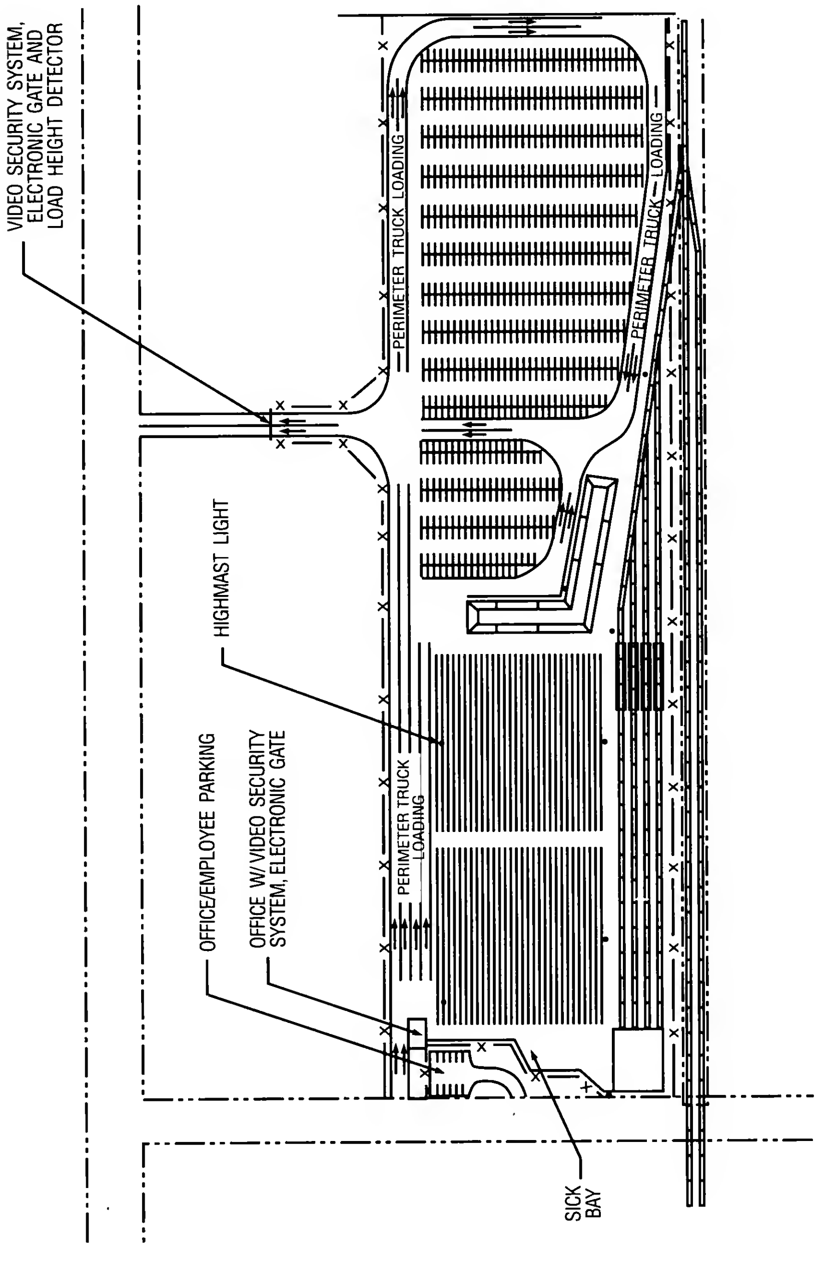

VIDEO SECURITY SYSTEM, ELECTRONIC GATE AND LOAD HEIGHT DETECTOR
PERIMETER TRUCK LOADING
PERIMETER TRUCK LOADING
HIGHMAST LIGHT
OFFICE/EMPLOYEE PARKING
OFFICE W/ VIDEO SECURITY SYSTEM, ELECTRONIC GATE
PERIMETER TRUCK LOADING
SICK BAY

Figure 2.

Chain link fence is one of the most cost efficient and effective types of fence. Fencing should consist of galvanized steel fabric with horizontal rails and tension wires. Fabric should be at least #9 gauge hot dip galvanized per ASTM A-392, Class 1, 2 inch mesh. Minimum height of fabric should be 8 feet. Tension wires, top and bottom, should be #7 gauge and conform to ASTM A-824. Posts should be steel pipe per ASTM A-120. Fence should be constructed in a manner that will not allow deformation to occur. Considerations for maximum fence strength include diameter of posts, depth planted, bracing, post spacing, fabric tension, and concrete footings.

Regardless of fence or wall type, the addition of barbed wire should be considered to compliment the structure. Wire should be attached to a V or 1/2 V rake, placed on every pole or no less than every 10 feet. There should be a minimum of three strands of wire per leg of V.

In high crime areas razor ribbon or concertina wire should be considered in addition to the barbed wire strands.

d) *Gates*

Gates should be designed to prohibit the entrance or exit of unauthorized vehicles and persons, and to control the traffic flow of trucks entering and exiting the facility. Gates are also used to control the entrance and exit of locomotives and rail cars.

Gates can be constructed from iron, tubular steel, flat steel, and chain link. They should be at least as high as the perimeter fence. The structural integrity should be reinforced and greater than the fence alone. They can be opened and shut by swinging, sliding, or rolling up. Drop bars should be solid steel. All gate hinges should be tack welded to gate posts to prevent gates from being lifted off.

Electronically controlled, motorized gates can be activated on site, from a remote location, or self activating underground wire using loops and a card reader system. A telephone or intercom will be required at the gate if it is monitored from a remote site. Gate stability is a primary concern if the facility has a fence alarm system.

e) *Tire Spikes*

Depressible spikes can be used at gate areas to control traffic flow and prevent unauthorized exit of vehicles from the facility. They should be well signed and considered a secondary system.

Heavy snow and ice may interfere with the operation of these units and available heating systems may be required, along with appropriate drainage to carry off melted snow and ice.

f) *Fence Alarm Systems*

Fence alarm systems detect the presence of a person or device against the fence. They sound an alarm, either audible, silent, or both when someone tries to climb, cut or jack up the fence.

These systems use point or line sensors, or fiber optic strands to discern impact. A circuit of electricity or light passing through the sensor or fiber is altered when the fence material deflects or breaks. A processing unit is required to respond to circuit changes and signal an alarm, either locally or to a remote location, via telephone modem.

The system should be installed in zones, the number of which to be determined by the size of the facility.

Terrain, environmental, and weather conditions should be taken into account when considering such a system.

g) *Closed Circuit Television*

A closed circuit television system acts as both a deterrent to theft and vandalism, and as a means of obtaining an accurate record for the investigation of criminal cases. They can be positioned for total or partial coverage of the facility. They can also work in conjunction with the gates, running continuously or activated only when the gates are utilized. Cameras should be capable of recording in color, onto a VCR recorder. The use of color cameras will require additional lighting, up to 10 foot candles.

h) *Card Access Systems*

A card access system is an effective method of monitoring or maintaining an inventory of all persons entering and leaving an auto facility.

Authorized persons are issued preassigned cards in advance. Upon entering or departing the facility, they activate the gate/card system with their cards. The monitoring/gate access decision making can be performed on site or from a remote location using a computer and phone modem. It is tied directly to the gates, authorizing and monitoring their functions.

The card access system consists of cards, card readers, processing controller, software, and a computer. Each card reader may contain its own microprocessor that permits memory and decision making at individually secured gates and doors. It should also include a battery backup system for use in the event of a power failure.

The card access system can be tied electronically to the fence alarm system, monitoring both functions.

i) *Signs*

Signs placed around the perimeter of the facility can deter trespassing. When placed within the facility they are beneficial in controlling traffic flow.

They should be located as not to obstruct the view of the drivers and other personnel. For easy recognition, they should be constructed in a manner similar to those recommended by the *Manual of Uniform Traffic Control Devices*. Preferably, signs should be made of aluminum with a reflective backing.

4.3.5.5 Buildings and Employee Accommodations

Accommodations should be provided for security guards if applicable. Parking for employees should be provided in a separate, secured area.

4.3.5.6 General Comments

A maintenance and system testing schedule for all electronic equipment should be developed and followed.

Emergency stand-by generators should be considered. This system will provide power for lighting, card readers, gates and/or the perimeter detection system if desired. It should be actuated automatically upon its sensing the loss of commercial external power to the facility.

Page 14-4-18. Article 4.5 Terminal Design for Bulk Fluids. Add the following new article:

4.5 TERMINAL DESIGN FOR BULK FLUIDS

4.5.1 Introduction

Bulk fluid terminals are specialized freight terminals which are used to transfer bulk lading from point of origination to rail cars, from rail cars to point of destination, or between rail and other modes of transportation. Some terminals may be designed purely for the transfer of commodities to other modes or directly to a customer, whereas other terminals may provide intermediate storage between modes, or storage on behalf of the customer.

This section is applicable to bulk liquids such as chemicals, petroleum, fertilizers, food-grade liquids and oils. Also, some dry bulk solids such as powders and granules, which have physical characteristics similar to a liquid, and are handled as fluids rather than as solids.

These commodities could be transported in single or multiple railcar blocks, or in unit train service. Some commodities, such as petroleum products, may be transported in railcars with interconnected piping to allow unloading and loading of several railcars from a single point.

Contingent upon the customer service to be afforded and the commodity to be handled, terminals may range in size and purpose from a single track, single car spot facility to a multiple track facility capable of unloading or loading unit trains. Individual customers may be served at a terminal or multiple customers may share the facility and its equipment. One or more different commodities may also be handled in the same terminal.

Factors affecting terminal design include number and types of materials to be handled, the size of shipment (i.e., unit train, ship, barge, multiple car, single car), the physical characteristics of the site, and the degree of processing and storage to be done on the site.

Although consideration herein is primarily directed to such common transfer terminals, design principles may be applicable to in-plant and other transfer facilities.

4.5.2 Site Selection

The site should be selected to accommodate both near and long term development of the terminal to handle the volumes of traffic projected for each commodity. Ease of access for customers and all modes of transportation involved are critical in selecting a site suitable for a terminal. Site selection and configuration should allow for economy in movement of materials, unloading and loading equipment, and transportation equipment.

Modification of an existing yard, particularly a team yard, may permit utilization of little used assets and use to advantage a site with good access. In other instances, selection of an active, new or undeveloped location may be prudent.

The following factors should be considered during the selection, planning and construction of the site.

4.5.2.1 Environment

Various chapters of this Manual discuss environmental considerations in detail. Environmental items relating to a site that typically impact terminal design that should be considered include:

1. Air pollution (vapor and dust control and collection)
2. Water pollution (rainfall runoff, spill containment, treatment facilities)
3. Spill containment (for liquids and solids)
4. Noise levels (impact on terminal employees and surrounding areas)
5. Light pollution (from terminal lighting, vehicles, equipment)
6. Proximity to archaeological and historic sites
7. Proximity to residential areas
8. Proximity to ecologically sensitive areas including wetlands

4.5.2.2 Size

The site selected for a terminal should have sufficient land area to allow future expansion and development of the terminal. Sizing of equipment and structures should allow for expansion and flexibility of operation.

The length of time allocated to discharge vessels, railcars, trucks and storage areas and the frequency of transportation service will impact the sizing of various elements of a terminal.

4.5.2.3 Access

4.5.2.3.1 Roads

Highways, streets and other roads to be used for access must provide an efficient route for customers. Routes to the site should be carefully studied for their ability to accommodate trucks and equipment that will serve the terminal. Road weight restrictions including seasonal restrictions, pavement widths, curves, intersections and existing traffic volumes and patterns should all be considered relative to the size and type of trucks and equipment that will use them.

Routes for trucks serving the terminal should also be carefully studied to determine whether they will pass or be near schools, hospitals, parks, community centers, residential areas, and other sensitive areas. Local ordinances may exist that prohibit truck traffic on certain roads. Also, site selection should consider public opposition that may prevent new or additional traffic on certain roads.

Site access for emergency vehicles should also be considered, incorporating specific access roads or gates into the site plan as necessary for use by emergency vehicles only.

4.5.2.3.2 Waterways

Water access should provide sufficient draft, maneuvering and turning basins, and berthing space for the size and type of vessels to serve the terminal.

4.5.2.3.3 Rail

Rail access should be designed to efficiently accommodate rail traffic serving the terminal's customers. The length of cars, locomotives and trains, frequency of switching movements serving the terminal, and the characteristics of existing mainline train movements and other operations, should be considered.

The availability of existing tracks or the ability to construct new tracks in yards or along running or main tracks to support the short and long term needs of the terminal should be considered.

4.5.2.4 Utilities

Utilities required for the site should be considered during the terminal site selection process. Water will be necessary for fire protection, employee washdown (i.e., showers, eye washout), dust control, equipment cleaning and employee facilities. Electrical power will be needed for commodity handling equipment, lighting, heating/cooling/ventilation equipment and other equipment. Sewage disposal is likely to also be needed.

4.5.2.5 Zoning and Permitting

Many governmental agencies have enacted laws which may impact the selection and construction of bulk fluid terminals. Proposals to locate this type of terminal in areas not properly zoned or near residential, commercial or recreational areas including schools and hospitals are frequently controversial to the public. Public hearings and other legal processes frequently become necessary when a zoning change or when a controversial site is selected.

Permits of some description are generally required at nearly all locations.

Schedules for placing a terminal in-service should consider the time associated with such hearings and legal processes and obtaining permits. In situations in which the timely completion of a terminal is critical, it may be prudent to select a site that will not arouse controversy.

4.5.3 Unloading and Loading Facilities

Unloading and loading facilities at terminals may vary from low-volume, single or multiple car and customer systems to high-volume systems for unit trains. Contingent upon the function of the terminal and the commodities to be handled, the transfer of commodities may be between railcar and truck, railcar and storage tank, railcar and water vessel, truck and storage tank, and/or truck and vessel. In any case, the facilities must be carefully designed to meet the needs of its customer or customers.

For low-volume terminals, small portable or fixed pump systems may be utilized, and similarly, small portable or fixed vacuum systems may be utilized for powders or granules. Some commodities being transferred between a railcar and truck in low-volumes may be handled using the truck's onboard pump or vacuum equipment. Intermittent unloading of commodities is also common in smaller terminals and will impact the equipment for the terminal.

For larger terminals, stationary, high-capacity equipment may be necessary.

In any situation, typical railcar and truck length should be determined for the installation of loading booms or unloading connections at the appropriate interval. Careful consideration must be given to the type of commodity and railcars, trucks, vessels and unloading/loading equipment to ensure compatibility. Also, a careful analysis of the equipment, piping, connections, storage tanks, and other facilities should be done to ensure that they are composed of materials that will not corrode or deteriorate when exposed to the commodity.

All equipment, including loading booms and unloading connections, must be retractable to enable it to clear railroad tracks pursuant to the guidelines found in the chapter for Clearances of the Manual.

4.5.3.1 Services

Certain commodities may require specialized services to effect their transfer between modes or to and from storage, such as electricity to power transfer machines, compressed air to move powders or granules between vehicles, steam or hot water to decrease viscosity of liquids, and nitrogen to purge railcars and pipelines. Provisions for these services at convenient locations along tracks or in other areas must be considered and incorporated in the design of the terminal.

Railcars, trucks and vessels, particularly those with special linings, may require specialized cleaning after each unloading or prior to use for other commodities. Specialized equipment, personnel, and facilities may be necessary to perform these functions to meet regulatory, customer and equipment owner needs and to protect equipment from damage and failure.

4.5.3.2 Walkways

Elevated walkways may be necessary to permit personnel to safely access the top of railcars and trucks for unloading and loading purposes. Retractable, telescoping or hinged walkway sections to reach the tops of railcars and trucks from elevated walkways parallel to the track or driveways, are common. Typical railcar and truck length should be determined to construct these sections at the appropriate intervals.

4.5.3.3 Sampling and Weighing

Sampling, weighing or metering provisions may be necessary for certain customers and commodities. The *AAR Scale Handbook* provides guidance on such facilities.

4.5.3.4 Environmental Facilities

It may be necessary and required by laws or regulations to construct spill containment systems such as dikes, paving and other appurtenances at unloading/loading areas and in commodity storage areas. To prevent contamination of the atmosphere, vapor or gas collection systems may be necessary or agency required. For some powders and granular commodities, dust collection or abatement systems may be necessary or required. Also, special treatment and pre-treatment facilities for the discharge of water may also be necessary.

Various chapters of the Manual discuss environmental considerations and design criteria in detail.

4.5.4 Commodity Storage

Most bulk fluid terminals require some level of storage capability to accommodate fluctuations in commodity demand, unloading and loading constraints between transportation modes, and blending of materials on-site.

The transfer and storage systems for a bulk fluid terminal should be designed to utilize gravity, minimize the handling of commodities as much as possible, and be of appropriate size or capacity to unload or load railcars, trains, vessels and/or trucks. Covered unloading or loading areas, stacks, silos or sheds may be desired to protect commodities from exposure and loss. Commodities composed of fine particle size, that are prone to become airborne by wind or other air movements, should be covered or otherwise protected.

4.5.5 Buildings

Buildings may be required for a variety of purposes. These could include offices and supporting facilities for employees, commodity storage, enclosure of commodity transfer areas, protection of boilers, water heaters and transfer equipment, security and any other function or item needed given weather and general site conditions.

Structures in bulk material handling terminals should be designed for durability and ease in cleaning. Electrical equipment and other sensitive equipment may require air conditioning and a dust-free environment. Lighting and ventilation must be designed to assure the safety of employees and allow the efficient execution of their duties. Clearances for railroad and mobile equipment should be considered. All structures must meet all applicable OSHA requirements and any local building and fire codes. Additional guidelines for structural design and construction are found in various chapters of the Manual.

Office buildings should be located for convenience near the entry to the terminal to allow monitoring of traffic in and out of the terminal, and to monitor the activities within the terminal itself. Separate offices and facilities may be necessary for outside contractors operating all or various portions of the terminal.

Certain customers and environmental regulations may require that commodity transfers be performed within an enclosure to protect the commodity from degradation or escape into the atmosphere.

Buildings for storage and servicing of transfer and other terminal equipment is typically required at most terminals. It is particularly critical that buildings be provided for equipment handling food grade commodities which require cleaning and protection from contamination. An appropriate work area as might be required for cleaning and maintenance of equipment, and a storage area with racks for hoses, fittings and other items for maintaining the equipment, should be provided in the building.

4.5.6 Security

In most instances, the commodities handled at bulk material terminals are of a unit value that security concerns address preservation of purity and protection of equipment.

At many sites, no sophisticated security measures are justified, other than to restrict points of entry to the terminal with perimeter fencing and a limited number of gates to allow ease of monitoring during operating hours and closure of the terminal during non-operation.

Area and perimeter lighting aids in deterring intruders and allows monitoring at night. Lighting levels should be such that shadowed areas are minimized.

Undergrowth and trees should be removed as needed around fences to prevent their use to breach or scale fences, and to allow improved visibility for monitoring the terminal perimeter.

4.5.7 Environment and Maintenance

Bulk material terminals should be designed in conformance with all federal, state and local environmental laws and regulations, and to allow easy maintenance of the infrastructure and equipment to minimize the potential and resulting impact of spills and site contamination. Various chapters of the Manual discuss environmental considerations in detail. Environmental items relating to the design of the terminal that should be considered include:

1. Air contamination (vapor and dust control and collection)
2. Water contamination (rainfall runoff, spill containment, treatment facilities)
3. Soils contamination
4. Noise levels (impact on terminal employees and surrounding areas)
5. Light pollution (from terminal lighting, vehicles, equipment)

Equipment and measures should be employed to control and/or collect airborne particles to prevent pollution of the atmosphere, dust explosions, adverse affects on employee health, loss of commodity, and deterioration of facilities and equipment from dust accumulation. The terminal's equipment and facilities should be constructed of materials resistant to deterioration from the commodities handled.

Areas where commodity spills are likely should be easily accessible for loaders and trucks to facilitate cleanup. Paving in areas where not structurally necessary may still be desirable to provide a barrier between commodities and the ground.

A drainage system should be provided which will effectively remove stormwater runoff to avoid deterioration of work surfaces, contamination of commodities, and minimize the impact upon unloading and loading operations. The systems should be designed to channel runoff to a central location for ease of containment, cleanup and/or treatment of spills, and should be constructed of materials that will not interact with any potential spill material. Similarly, the drainage system should be designed to allow the easy removal of any residue or sedimentation to prevent any potential interaction with any other materials spilled.

4.5.8 Terminal Configuration

The terminal should be configured to provide the most efficient movement of commodities, transfer equipment and transportation vehicles. Security, safety and environmental facilities and appurtenances appropriate to the commodities handled should be considered in the design. Service facilities and utilities should be strategically located to allow easy access without conflicting with other operations, activities or movements within the terminal.

The length of time allocated to discharge vessels, railcars, trucks and storage areas and the frequency of transportation service will impact the configuration of various elements of a terminal.

See Figures 1 and 2 for examples of bulk fluid terminals.

4.5.8.1 Tracks

Bulk fluid terminals may be served by tank cars and/or covered hopper cars of varying lengths and capacities contingent upon the commodity being carried. At low-volume terminals, railcars tend to be switched individually or in relatively short blocks or "cuts," whereas at high-volume or large terminals, larger blocks or unit train movements may be employed. Track lengths, switching leads and ladders must be designed pursuant to the type of operation or service planned.

The overall design of the terminal, including the track configuration, must provide adequate room to accommodate driveways for unloading and loading equipment, service equipment and inspections. Also, the design must minimize any conflicts between trucks, unloading and loading equipment, and rail movements to allow the terminal to operate as efficiently as possible. Intermittent unloading of commodities is also common in smaller terminals and will impact the configuration of the terminal.

For terminals to receive individual or short cuts, relatively short track lengths for unloading and loading may be prudent to allow switch engines easy access to individual railcars. Multiple, short tracks may be necessary to enable switching without having to halt or await the completion of other railcar unloading or loading activities, or move partially loaded railcars.

For terminals to receive longer cuts or unit train service, longer unloading or loading, lead and storage tracks lengths will be necessary.

In terminals where access is needed only on one side of a track for unloading or loading, tracks may be configured in pairs with services and lighting required placed between the tracks. Paired tracks must have track centers that provide clearances which conform with the guidelines presented in the clearance chapter of the Manual and governmental regulations.

Covered hopper cars can be expected to range from 100 to 125 tons, therefore, trackage and subgrade construction should be of a design to accommodate heavy axle loadings.

Blue flag protection should be provided at unloading and loading locations in the terminal, or any other location where employees will be working on top of, beneath, or inside of railcars.

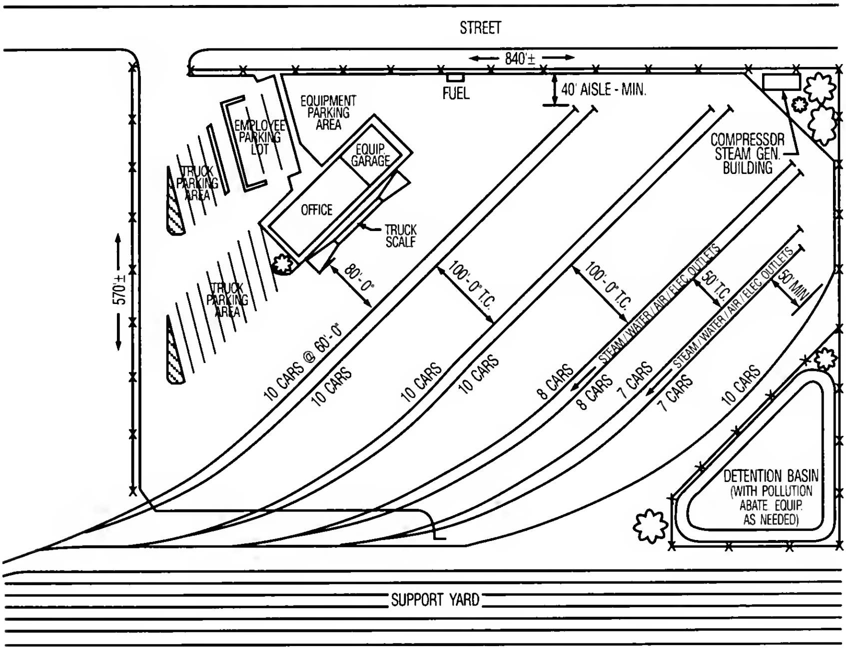


Figure 1. Bulk Fluid Transfer Terminal—Single End Switching. Capacity: 80 cars spotted. 11± plus storage/support yard.

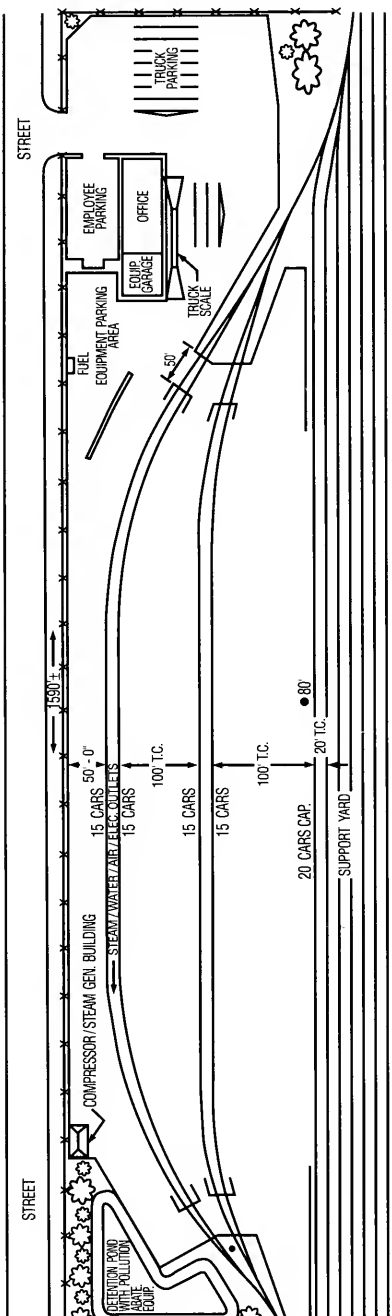


Figure 2. Bulk Fluid Transfer Terminal—Double End Switching. Capacity: 80 cars spotted. 11± plus support yard.

4.5.8.1.1 Track Geometry—Non-Unit Train Operation

Trackage for non-unit train terminals varies widely with the type of terminal. In general, the following track design standards are recommended:

- Maximum curvature—12 degrees–30 minutes
- Minimum turnout—Number 8
- Lead tracks—Length should be as long as the longest storage track
- Gradients—preferably flat or sloping toward the end of track at a grade not to exceed 0.1%; grades for storage tracks should be such that application of hand brakes is not necessary; a slight ascending grade should be included at either end of the storage tracks to prevent roll-outs

In some terminals where cars are moved by gravity, the principles of hump yard design can be used. These principles can be found in Part 2 of this chapter.

4.5.8.1.2 Track Geometry—Unit Train Operation

For trackage to accommodate unit trains, curvature and gradients should be designed with unit train dynamics considered. The following criteria are recommended:

• Maximum curvature on loops and lead tracks	7 degrees–30 minutes
• Maximum gradient on approach to loop	1% compensated
• Gradient on unloading loop	Level or slight ascending grade
• Minimum turnout for loop and lead	Number 10
• Maximum rate of change for vertical curves	0.12 per 100 foot station in sags; 0.20 per 100 foot station in crests

The AAR Train Performance Calculator or other similar train dynamics simulators can be run to verify train performance over a proposed design.

Trackage affecting the operation of unloading and loading equipment, train and other material handling equipment should take into account the requirements and recommendations of the equipment manufacturers.

4.5.8.2 Driveways

At locations where the transfer of commodities between rail and trucks is to occur, driveways of sufficient width must be provided on at least one side of each unloading or loading track to permit a truck to park. The width allowed for each truck should be 12 feet plus any width required to angle a truck relative to the track and railcar. Additional width must be provided to allow other vehicles to safely pass the parked truck and for sufficient space to position any transfer equipment. Adequate turning radii for trucks must be provided to promote the unobstructed and efficient flow of traffic and equipment.

Parking for employees and visitors should be provided in a separate area from the terminal operations to minimize traffic congestion and promote security of the terminal's equipment and supplies. Parking should be located in close proximity to the office building, but positioned so that pedestrians and vehicles are clear of the circulation of trucks, equipment and other vehicles.

Paving must be designed to support the loads anticipated from fully loaded tractor trailer trucks and transfer equipment. Selection of pavement materials must be appropriate to the service. Some commodities may damage pavement if spilled, such as petroleum products in contact with bituminous concrete. Crushed stone or gravel may be appropriate at smaller terminals, however, some aggregate particles may interact with commodities or its dust may contaminate commodities. Also, spills onto stone areas can be difficult to clean up and could allow contamination of ground beneath the stone paving.

For guidelines concerning road and pavement design, see the "AASHTO Guide for Design of Pavement Structures", published by the American Association of State Highway and Transportation Officials, Chapter 4. Paving materials and construction methods for a given area are typically specified to meet state or local highway authority specifications.

4.5.8.3 Truck Scale

Many customers require that commodities and drayage be weighed at the terminal. The location of the scale should be carefully planned to allow trucks easy access to the scale without adversely affecting activities elsewhere in the terminal when entering and exiting the terminal. The location of the scale should also permit trucks to easily return to an unloading area if necessary to "top off" their load.

It may be desired to position the scale in close proximity to the terminal office to allow scale equipment to be placed within the building for protection and use by office personnel.

Proposed 1996 Manual Revisions to Chapter 15—Steel Structures

Page i Foreword

Revise the opening of the foreword to read as follows:

Parts 1 and 3 through 7 formulate specific and detailed recommendations for the design, fabrication, erection, maintenance, inspection, and rating of steel railway bridges for:

- Spans up to 400 ft.
- Standard gage track
- Normal North American passenger and freight equipment, and
- Speeds of freight trains up to 70 mph and passenger trains up to 90 mph

The requirements, however, apply to spans of any length, but special provisions for spans longer than 400 ft. should be added by the company as may be required. Part 8 covers miscellaneous items. Part 9 is a commentary, including bibliography, for explanation of various articles in the preceding parts.

Grateful acknowledgement . . . (no change)

Part 1—Design

Page 15-1-3. In Article 1.2.1(a), the third paragraph, eliminate the word "welded." The paragraph will now read:

"For bridge construction, the material shall not be rimmed or capped steel."

Page 15-1-4. In Table 1.2.1A eliminate all references to Footnote 4. (3 locations)

In Table 1.2.1A eliminate Footnote 4.

Page 15-1-40. Article 1.10.2(c)—eliminate the word "fillet." The Article will now read:

"(c) Intermittent welds"

Article 1.10.2(e)—change to read:

"(e) Partial joint penetration groove welds transverse to the direction of stress."

Page 15-1-49. Change the second paragraph of 1.14.8(c)3 to read:

"If the average energy valve of the three remaining specimens is below the specified minimum requirement, or if the energy valve for more than one of the three specimens is below the specified minimum requirement, or if the energy valve for one of the specimens is less than two-thirds (2/3) of the specified minimum requirement, one retest from the same plate shall be permitted and the energy valve of all three retest specimens after discarding the highest and lowest valves, shall equal or exceed the specified minimum requirement."

Part 3—Fabrication

Page 15-3-13. Add the following sentence to Paragraphs 3.5.5(b) and 3.5.5(c): "If rejectable discontinuities are found, the provisions of AWS D1.5 for additional testing shall apply."

Part 5—Special Types of Construction

Page 15-5-8. Replace the entire existing Article 5.2.9.2 with the following:

(a) Flanges of welded plate girders shall be made using only one plate in each flange (i.e., without cover plates).

(b) Partial length cover plates may be used on rolled beam spans under the following conditions:

1. Partial length cover plates preferably shall be limited to one on any flange. The maximum thickness of the cover plate (or total thickness of all cover plates) on a flange shall not be greater than 1.5 times the thickness of the flange to which the cover plate is attached.
2. Cover plates may be wider or narrower than the beam flange to which they are attached.
3. Any partial length cover plate shall extend beyond the theoretical end by the terminal distance, or it shall extend to a section where the stress range in the beam flange is equal to the allowable fatigue stress range for base metal adjacent to or connected by fillet welds, whichever extension is greater. The terminal distance is 2 times the nominal cover plate width for cover plates not welded across their ends, and 1.5 times for cover plates welded across their ends. The width at ends of tapered cover plates shall be not less than 3 inches. All welds connecting the cover plate to the flange in its terminal distance shall be of sufficient size to develop a total stress of not less than the computed stress in the cover plate at its theoretical end.

Part 6—Movable Bridges

Page 15-6-2. Revise Article 6.1.2 as follows:

6.1.2 Abbreviations

(a) The following abbreviations are used herein:

AAR	Association of American Railroads
ABMA	American Bearing Manufacturers Association
AGMA	American Gear Manufacturers Association
AISE	Association of Iron and Steel Engineers
AISI	American Iron and Steel Institute
ANSI	American National Standards Institute
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
AWG	American Wire Gage
IEEE	Institute of Electrical and Electronics Engineers
IPCEA	Insulated Power Cable Engineers Association
NEC	National Electrical Code
NEMA	National Electrical Manufacturers Association
NFPA	National Fluid Power Association
SAE	Society of Automotive Engineers

Page 15-6-2. Revise paragraph (b) of Article 6.1.4 Machinery and Hydraulic Drawings, as follows:

(b) The Contractor shall make hydraulic control circuit and piping diagrams, hydraulic power unit layouts, and all assembly and detail drawings including the electrical schematic wiring diagrams and conduit diagrams that are needed for the complete hydraulic system. The drawings shall be so complete that the hydraulic components can be replaced without having the original stock numbers of the equipment. The drawings shall also conform to the requirements of the NFPA Standards.

Page 15-6-4. Revise paragraph (c) of Article 6.1.12 Wiring Diagrams, Operator's Instructions, Electrical, Hydraulic and Mechanical Data Booklets, and Lubrication Charts, as follows:

(c) The Contractor shall also furnish six bound copies of a similar booklet for the hydraulic equipment, which shall conform to the requirements of the NFPA Standards.

Part 15-6-24. Article 6.5.1 Fits and Surface Finishes. Revise the fit and finish for "Turned bolts in finished holes," as follows:

Fit / LT 1 Finish / 63

Page 15-6-26. Revise paragraph (a) of Article 6.5.22 Anti-Friction Bearings, as follows:

(a) Anti-friction bearings shall be so sized that under the loads and resistances specified in Section 6.3, and at the average running speed at which the bearing is applied, the B-10 life shall be at least 40,000 hours. (B-10 life shall be as defined by the ABMA and shall be the time for which 90% of a group of identical bearings will survive under the given loading conditions).

Page 15-6-28. Revise paragraph (b) of Article 6.7.5.35 Control Console, as follows:

(b) The control console shall be located so as to afford the operator a clear view in all directions. The console shall be of cabinet type construction with a horizontal front section about 36 in. above the floor and an inclined rear instrument panel set at such a slope that the meters can be read from average eye level without parallax and without reflection from the glass instrument cover. The console plan dimensions and the arrangement of equipment shall be such that all control devices are within easy reach. The top of the console shall be a laminated phenolic compound not less than 1 in. thick, with edges beveled and neatly finished. Where specified by the Company, the top of the console may be of No. 10 U.S. Standard gage stainless steel with a non-reflecting finish. The horizontal and sloping sections of the top shall be accurately cut to ensure a close fit.

Page 15-6-60. Revise paragraph (d) of Article 6.7.5.38 Electric Wires and Cables, as follows:

(d) Insulated wires for connections to the motor resistance grids shall contain no asbestos, be flexible stranded copper conductors with a minimum Underwriters Laboratory operating temperature of 150°C.

Page 15-6-32. Delete existing Article 6.5.25 and replace it with new Article 6.5.25, as follows:

6.5.25 Bolts and Nuts

(a) Bolts for connecting machinery parts to each other or to steel supporting members shall conform to one of the following types:

1. Finished, high-strength bolts.
2. Turned bolts, turned cap screws, and turned studs.
3. High-strength turned bolts, turned cap screws, and turned studs.

(b) Finished high-strength bolts shall meet the requirements of ASTM A449. High-strength bolts shall have finished bodies and regular hexagonal heads. Holes for high-strength bolts shall be not more than 0.01 inch larger than the actual diameter of individual bolts and will require drilling holes to match the tolerances for each bolt. The clearance shall be checked with 0.011-inch wire. The hole shall be considered too large if the wire can be inserted in the hole together with the bolt.

(c) Turned bolts, turned cap screws, and turned studs shall have turned shanks and cut threads. Turned bolts shall have semi-finished, washer-faced, hexagonal heads and nuts. Turned cap screws shall have finished, washer-faced, hexagonal heads. Finished shanks of turned fasteners shall be 1/16 inch larger in diameter than the diameter of the thread, which shall determine the head and nut dimensions. The shanks of turned fasteners shall have Class LT1 fit in the finished holes in accordance with ANSI Standard B4.1. The material for the turned shank fasteners shall meet the requirements of ASTM A307, Grade A.

(d) High-strength turned bolt, turned cap screws, and turned stud details shall be as specified in Article 6.5.25(c), except that the material shall meet the requirements of ASTM A449.

(e) Elements connected by bolts shall be drilled or reamed assembled to assure accurate alignment of the hole and accurate fit over the entire length of the bolt within the specified limit.

(f) The dimensions of bolt heads, nuts, castle nuts, and hexagonal head cap screws shall be in accordance with ANSI Standard B18.2, Square and Hexagon Bolts and Nuts.

(g) Heads and nuts for turned bolts, screws, and studs shall be heavy series.

(h) The dimensions of socket-head cap screws and socket flathead cap screws shall conform to ANSI Standard B18.3. The screws shall be made of heat-treated alloy steel, cadmium-plated, and furnished with a self-locking nylon pellet embedded in the threaded section.

(i) Threads for bolts, nuts, and cap screws shall conform to the coarse thread series and shall have a Class 2 tolerance for bolts and nuts or Class 2A tolerance for bolts and Class 2B tolerance for nuts in accordance with ANSI Standard B1.1, Unified Inch Screw Threads.

(j) Bolt holes through unfinished surfaces shall be spotfaced for the head, nut, and washer, square with the axis of the hole.

(k) Unless otherwise called for, bolt holes in machinery parts or connecting these parts to the supporting steel work shall be subdrilled at least 1/32 inch smaller in diameter than the bolt diameter. They shall be reamed for the proper fit at assembly or at erection with the steel work after the parts are correctly and finally assembled and aligned and the supporting steel work subdrilled.

(l) Holes in shims and fills for machinery parts shall be reamed or drilled to the same tolerances as the connected parts at final assembly.

(m) Positive locks of an approved type shall be furnished for nuts, except those of ASTM A449 bolts which are tensioned at installation to at least 70 percent of their required minimum tensile strength. If double nuts are used, they shall be used for connections requiring occasional opening or adjustment. If lock washers are used for securing, they shall be made of tempered steel and shall conform to the SAE regular dimensions. The material shall meet the SAE tests for temper and toughness.

(n) High-strength bolts shall be installed with a hardened plain washer meeting ASTM F436 at each end.

(o) Wherever possible, high-strength bolts connecting machinery parts to structural parts or other machinery parts shall be inserted through the thinner element into the thicker element.

(p) Cotters shall conform to the SAE standard dimensions and shall be made of half-round stainless steel wire, ASTM A276, Type 316.

(q) Anchor bolts connecting machinery parts to masonry shall be ASTM A307, Grade A material, hot-dipped galvanized per ASTM A153. Bolts shall be as shown on the masonry drawings. Anchor bolts for new construction preferably shall be cast-in-place and not drilled. The Engineer shall specify the material and loading requirements for the given design condition. When these fasteners connect a mechanical component directly to the concrete, filler material must be put in the annular area between the bolt and the bolt hole in the machinery component. The filler material may be a non-shrink grout, tin based babbitt metal, or zinc.

(r) Fasteners shall be of North American manufacture and shall be clearly marked with the manufacturer's designation.

Page 15-6-68. Revise Article 6.8.14, as follows:

6.8.14 Bolts and Holes

(a) Bolts for minor machinery parts may be unfinished and shall have drilled or reamed holes not more than 1/16 in. larger diameter than the bolts if approved by the Engineer.

(b) All fasteners and their mounting holes not included in (a) shall conform to the requirements of Article 6.5.25.

Page 15-6-70. Revise Article 6.9.1 Erection of Machinery, by the addition of new paragraph (d), as follows:

(d) Open gearing shall be aligned such that backlash is within tolerance so that at least the center 50% of the face width of each pair of meshing teeth is in contact. The cross mesh shall not exceed .01 inch per 6-inch face width. Open gear measurements shall be submitted to the Engineer for approval. The measurements shall include backlash, cross mesh alignment, tooth valley gap and face contact. The type of bluing or lubricant used for face contact measurements shall be submitted to the Engineer for approval prior to any measurements. These measurements shall be performed at a minimum of eight (8) equally spaced span positions ranging from fully open to fully closed.

Existing paragraph (d) to be designated as (e).

Part 7—Existing Bridges

Page 15-7-4. Article 7.2.1.4 Welding

Change the designation of existing Article 7.2.1.4, "Welding" to 7.2.1.5 "Welding."

Add the underlined words or phrases (see text below) to Article 7.2.1.5(c).

Add new Article 7.2.1.5(e)—(see text below)

Add new Article 7.2.1.5(f)—(see text below)

The revised article 7.2.1.5 will read as follows:

7.2.1.5 Welding

(a) Electric arc welding may be employed subject to the approval of the Engineer.

(b) In general, welds shall not be assumed to act together with rivets or bolts.

(c) Where welds are added to existing riveted *or bolted* connections, the welds shall be designed to transmit the entire force, except that in such members where the existing material carries the entire dead load force, the welds shall be designed to carry the entire live load force in the member. Where some of the existing rivets in a member are loose or defective, they *shall* be replaced with high-strength bolts property installed, unless otherwise directed by the Engineer, and such bolts may be considered to carry the dead load stress of the replaced rivets provided they are installed prior to the welding. *Loose rivet heads shall not be welded.*

(d) Welding shall be in accordance with the applicable sections of Part 1, and may be used only where specifically permitted.

(e) When welding existing material where mill scale, rust, and dirt are present, and standard surface preparation cannot be accomplished, low hydrogen electrodes shall be used.

(f) When difficult-to-weld material must be welded to effect a repair, use of global pre-heats and post-heats shall be considered. Refer to Alternative Pre-Heat Requirements of AWS.

Page 15-7-4. Add new Article 7.2.1.7 "Repair of Cracks and Defects" to read as follows:

7.2.1.7 Repair of Cracks and Defects

(a) An actively propagating fatigue crack, either load-induced or distortion-induced, may be temporarily repaired by drilling a hole in the member to encompass the crack tip, provided the remaining net section of the member has sufficient stress-carrying capacity. The hole size shall be at least equal to the thickness of the material, but not less than 3/4 in. (19 mm) diameter. Permanent repairs shall consist of measures to reduce the stress range in the case of load-induced fatigue cracking, and to eliminate the causes of the distortion in the case of distortion-induced fatigue cracking.

(b) Defects from Category 1 Damage, such as gouges, nicks, burrs, etc. on the surface of fracture critical members shall be repaired by grinding smooth or peening. No weld repair of such surface defects shall be permitted.

Page 15-7-9. Revise Article 7.3.1.1 Normal Rating, as follows:

Revise the fourth sentence in Article 7.3.1.1(a) to read as follows:

"Allowable stresses for normal rating shall be those specified in Section 1.4, supplemented by Article 1.3.14.3."

Insert the following paragraph as Article 7.3.1.1(c):

"If the normal rating is greater than the maximum rating, the lesser rating shall govern."

Page 15-7-9. Revise Article 7.3.1.2 Maximum Rating, as follows:

Insert the following sentence in Article 7.3.1.2(a):

". . . specified in Article 7.3.4.3. The provisions of Article 7.3.4.2 Fatigue need not be considered when determining Maximum Rating."

Insert the following sentence in Article 7.3.1.2(b):

". . . significantly shortened. See Commentary Article 9.7.3.1.2."

Page 15-7-11. Article 7.3.4.2 Fatigue.

Revise Article 7.3.4.2(b) to read as follows:

"Members with riveted or bolted connections with low slip resistance, subject to repeated stress fluctuations: the requirements of Category D of Article 1.3.13 shall be considered with a variable amplitude fatigue limit of 6 ksi up to 100 million cycles. Where the Engineer can verify that the fasteners are tight and have developed a normal level of clamping force, Fatigue Category C may be used provided the Root-Mean-Cube (RMC) stress range has not and will not exceed 9 ksi. If Category C is used, the variable amplitude fatigue limit is 6 ksi, up to 100 million cycles (see Figure 9.7.3.4.2B)."

Revise the first sentence in Article 7.3.4.2(c) to read as follows:

"Riveted and bolted connections and members that do not satisfy the requirements of Article 7.3.4.2(b): these requirements may be waived at the discretion of the Engineer if the Root-Mean-Cube (RMC) stress range is less than 9 ksi and if the connections or members will retain their structural adequacy in the event one of the elements cracks."

Revise Article 7.3.4.2(d) to read as follows:

"Wrought iron riveted connections shall be considered to have Category D fatigue strength."

Page 15-7-12. Revise Article 7.3.4.2 Fatigue by adding new paragraph (h), as follows:

(h) Any decision to not use Paragraph 1.3.13(i) (reduced permissible stress range for fracture critical members listed in Table 1.4.11) must be made by the Engineer of the railroad. The decision depends on the adequacy of inspection practices to identify fracture critical members and to detect a flaw or crack before serious damage from uncontrolled propagation will occur, together with the knowledge of actual loads and load history, as opposed to theoretical loads.

Page 15-7-14. Article 7.3.4.3 Allowable Stresses for Maximum Rating.

Revise the first sentence in Article 7.3.4.3(c) to read as follows:

"Members subject to both axial compression and bending stresses shall satisfy the following requirements: . . ."

Revise the first sentence in Article 7.3.4.3(d) to read as follows:

"For members subject to both axial tension and bending, the total of the axial tensile stress and the combined bending tensile stresses about both axes shall not exceed K."

Part 9—Commentary and Bibliography

Page 15-9-33. Insert the following new Articles in Part 9:

9.7.3.1.1 Normal Rating

The intent of the normal rating is to limit the stresses in the structure to those for which it would be designed given the yield strength of the steel in question and the design specifications of Part 1. *The normal rating* will *ensure* a consistent factor of safety and prolong the useful life of the structure.

The allowable rating stresses, when wind loads are included, can be increased to 25% greater than basic allowable stresses, but in no case greater than the allowable stresses for Maximum Rating. The 25% increase is included so that, for members such as truss chords where wind loading may be significant, the Normal Rating will not be less than the loading for which the member was designed."

9.7.3.1.2 Maximum Rating

Maximum rating recognizes that loads producing stresses higher than design values may be imposed on a structure. However, to maintain a consistent factor of safety and to reduce the effects of fatigue, it is recommended that loads up to the maximum rating be allowed only infrequently.

Paragraph 7.3.1.2(b) permits the Engineer to authorize more frequent maximum rating loads with the caution that the useful life of the structure will be thereby reduced. If frequent maximum rating loads are contemplated, it is appropriate that *either a more detailed inspection be made of fracture critical members or* a fatigue analysis be conducted per Articles 7.3.4.2 and 9.7.3.4.2 to predict the remaining useful life of the structure and preclude the continued application of loads beyond the *stage* where the potential *for* member failure is high. *Another alternative is to predict the theoretical remaining useful life and when this predicted life has expired, continue using the structure by making more detailed inspections of fracture critical members.*

It should also be remembered that maximum rating stress *results in* a reduced factor of safety.

Page 15-9-33. Insert the following new Article in Part 9:

9.7.3.4.1 Computation of Stresses

The provisions for intermediate stiffener spacing in Article 1.7.8(a) are derived from the equations for elastic and inelastic buckling of a flat web under shear stress, using suitable reduction factors. See Article 9.1.7.8. Those equations are critical load solutions for thin flat plates based on small deflection theory and do not consider post-buckling conditions in the web plate. The detailed analysis referred to in Article 7.3.4.1(a)2 is a more refined elastic/inelastic critical load analysis of a flat plate subjected to shear and bending (52, 53). The Engineer is advised to apply a reduction factor to the computed critical load to account for web plate out-of-flatness and other imperfections.

These comments do not consider the effect of stiffeners to support the top flange.

Page 15-9-33. Article 9.7.3.4.2 Fatigue.

Revise the fourth paragraph in Commentary Article 9.7.3.4.2 to read as follows:

"The fatigue resistance of members with riveted or other mechanically fastened connections with low slip resistance is defined by Category D as a result of review of available test data (42, 49, 50, 54, 55). The most recent research indicates a variable amplitude fatigue limit of 6 ksi, extending to at least 100 million cycles (54). Referring to Figure 9.7.3.4.2B, it is apparent that nearly all test data on riveted joints with normal levels of clamping force fall to the right of the line defined by Category C between 6 and 9 ksi. The existing test data (49, 50, 54, 55) show failures at high numbers of cycles below the constant stress range fatigue limit for Category C (10 ksi), but above the variable amplitude stress range fatigue limit value of 6 ksi. Hence, any evaluation using Category C must extend on to 6 ksi. For stress ranges above 9 ksi the test results for riveted connections typical for railroad bridges fall to the right of the line defining Category D."

Revise the fifth paragraph in Commentary Article 9.7.3.4.2 to read as follows:

"It is reasonable to permit a higher fatigue stress range for Root-Mean-Cube (RMC) stress ranges below 9 ksi if the connection or member in question has tight riveted joints. Where the rivets are tight and rivet holes are smooth, having been correctly drilled or subpunched and reamed, a further refinement in the allowable stress range is permissible. A line on the rivet S-N plot extending from Category C at 7.65 ksi to 6 ksi at 100 million cycles may be used in lieu of the horizontal line at 6 ksi (54, 55). This refinement does not apply to punched holes."

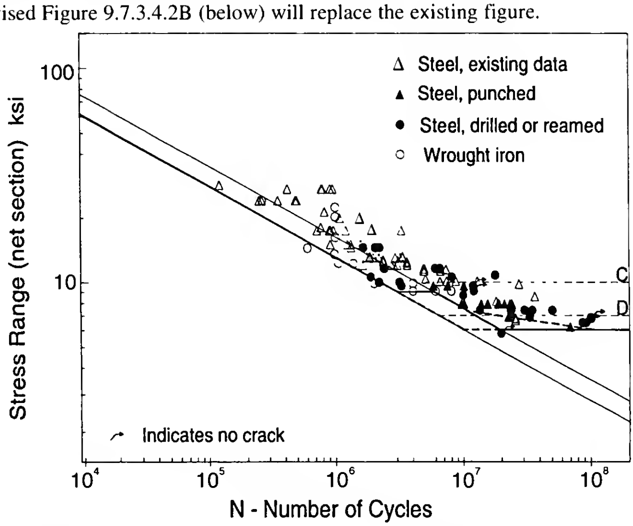
Revise the fourth sentence in the seventh paragraph in Commentary Article 9.7.3.4.2 to read as follows:

"Where the constant amplitude stress range exceeds 9 ksi, test results indicate that not much time elapses between easily detectable cracking and member failure."

Revise the ninth paragraph in Commentary Article 9.7.3.4.2 to read as follows:

"Wrought iron riveted connections exhibit a fatigue strength represented by Category D (54, 55)."

A revised Figure 9.7.3.4.2B (below) will replace the existing figure.



For riveted bridge components

For design only:

$N = 2.183 \times 10^9 S_r^{-3}$ $S_r > 9$ ksi

For evaluation:

$N = 2.183 \times 10^9 S_r^{-3}$ $S_r > 9$ ksi

$N = 4.446 \times 10^9 S_r^{-3}$ $9 \text{ ksi} > S_r > 6$ ksi

Fatigue limit: $(S_r)_{fl} = 6$ ksi

For optional evaluation of drilled or reamed bridge components:

(see 9.7.3.4)

$N = 2.183 \times 10^9 S_r^{-3}$ $S_r > 9$ ksi

$N = 4.446 \times 10^9 S_r^{-3}$ $9 \text{ ksi} > S_r > 7.65$ ksi

$N = 2.465 \times 10^{15} S_r^{9.5}$ $7.65 \text{ ksi} > S_r > 6$ ksi

Fatigue limit: $(S_r)_{fl} = 6$ ksi

Figure 9.7.3.4.2B. Riveted Bridge Components.

Page 15-9-43

Insert the following into the Bibliography:

52. Galambos, T.V., *Guide to Stability Design Criteria for Metal Structures*, 4th Edition, John Wiley and Sons, 1988, pages 89–108.

53. Salmon, C.G., and J.E. Johnson. *Steel Structures: Design and Behavior*, Third Edition, Harper and Row, Inc., 1990, pages 676–689.

54. Zhou, Y.E., B.T. Yen, J.W. Fisher, and R.A.P. Sweeney. Examination of Fatigue Strength (Sr-N) Curves for Riveted Bridge Members, Proceedings of the 12th Annual International Bridge Conference, Engineers' Society of Western Pennsylvania, 1995.

55. Akesson, B., and B. Edlund. Fatigue Life of Riveted Railway Bridges, Proceedings of IABSE Symposium on Extending the Lifespan of Structures, San Francisco, 1995, IABSE, Zurich, Switzerland, 1995, Vol. 2, Pgs. 1079–1984.

Proposed 1996 Manual Revisions to Chapter 17—High Speed Rail

Part 2—Track Structures and Train Interactions (under development)

Page ii. Table of Contents. Add the following outline of Part 2, currently under development.

2.5.2 Bridges
2.5.3 Tunnels
2.5.4 Stations Including Parking Facilities
2.5.5 Operation Center/Management
2.5.6 Maintenance of Way, Include Signal and Communications
2.5.7 Culverts
2.5.8 Retaining Walls
2.5.9 Miscellaneous
2.6 Clearances
2.6.1 General
2.6.2 Fixed Objects
2.6.3 Movable Objects

Add new Part 3, as follows:

Part 3—Vehicle, Control & Propulsion Systems Considerations

3.1 INTRODUCTION

This Section describes vehicle, control and propulsion system considerations that apply to high speed rail operation in the North American environment. While the success and commercial maturity of existing European and Japanese high speed rail technologies is recognized, certain modifications will be necessary to address North American operational practices and safety requirements. This Section discusses the necessary modifications and features, together with references to other sections of AREA "Manual for Railway Engineering" where appropriate.

3.2 GENERAL REQUIREMENTS

High speed rail rolling stock and control systems must be compatible with their high speed infrastructure. Rolling stock must also be suitable for operation over existing rail lines at conventional speeds. Vehicle control and propulsion systems must be compatible with the installed signal and communications systems on a route-specific basis.

3.2.1 General

While regulatory requirements for high speed rail operations in North America continue to evolve, it is expected that many of the current requirements regarding vehicle and control system designs will remain applicable. Areas that have required modification or regulatory waiver for high speed demonstration service in the U.S. have included car body structural strength and crashworthiness, safety appliances, and hand brakes (or parking brake arrangements).

High speed rail systems must be designed and implemented in accordance with federal and local regulatory and safety requirements applicable to the project. The designs and systems may incorporate an incremental approach with accommodation for intermediate speed ranges up to 125 mph (201 kph), use of non-electric/fossil fuel propulsion systems, and inclusion of tilt-suspension systems.

3.2.2 Interface Requirements

High speed rail operations involve a number of physical, operational, functional and electromagnetic compatibility (EMC) interfaces. Some are critical to system safety and all contribute to reliable system operation. Formal identification and control of these interfaces is mandatory and should be addressed in the form of a dedicated system integration effort.

For newly-built or otherwise dedicated high speed trackage, control of vehicle-infrastructure interface requirements can be based upon the technical specifications of the applicable proprietary technology. For existing trackage shared with other "conventional" rail operations, additional care must be taken to ensure full compatibility of the high speed vehicles and operating systems with current or evolving track and control installations in a manner acceptable to all user agencies.

3.2.3 Modification to Service—Proven Technology

Current European and Japanese high speed systems have generated good records of performance and have been proven in service. The application and proprietary details of these technologies vary in accordance with the railroad operating practices and the physical and economic constraints of the home country.

Past history in the application of off-shore rail technologies to the North American environment has indicated the need to review and alter even service-proven designs in light of North American service requirements. Changes have been made as necessary to accommodate the climatic, operating and maintenance environment and include car structural strength, brake system capacity, and other safety and safety-assurance functions. It is likely that other existing high speed rail technologies would require similar modifications.

3.3 DESIGN REQUIREMENTS

3.3.1 General

The purpose of this section is identification of issues and interfaces that should be addressed in development and design of high speed and very high speed rail systems. The following comments apply to these systems defined as specified in Paragraph 3.5.1.2 by using the generic term "high speed". The paragraphs will also cite other portions of this Manual as well as additional standards issued by relevant authorities. While very high speed rail operations are mature technologies in other countries, they are relatively new in North America, particularly in the very high speed range. There is only one U.S. high speed rail operation, along Amtrak's Northeast Corridor, that is confined to an improved, existing right-of-way and that uses the available infrastructure and systems. Consequently, a comprehensive body of suitable North American technical standards does not exist. Some current requirements may be incompatible with aspects of technologies under consideration, and the development of suitable standards is a desirable objective. The foreign technologies have been developed,. built and operated as systems encompassing all aspects of route geometry, right-of-way, traction power, signal and communications, vehicles, shops, terminals, maintenance practices, supervisory control and data acquisition (SCADA), and operations.

Application of more advanced foreign systems standards to development of new services may involve adaptation for North American operation. It may be preferable to modify or adopt such standards for new projects to ensure that system integrity is maintained, since they are already compatible being directly evolved from relevant technology. When high speed systems are evaluated for application in North America, the underlying standards, operating environment, and design criteria must also be considered as part of the technology transfer process.

3.3.2 Design Criteria

Current AAR, AREA and FRA standards and regulations should be investigated to identify applicable requirements. The standard practices and safety codes established by engineering and technical governing bodies (e.g. ASME, IEEE, NESC, etc.) are typically incorporated by reference rather than by quoting specific provisions. Applicable sections should be identified and dated and divergences should be explicitly stated. These codes and recommended practices normally comprise a portion of design requirements. Local, state or regional codes and requirements should also be reviewed for relevant items appropriate to the system. These may address localized effects of the system, such as noise, speed, safety, aesthetics, vibration, land use, accessibility and service. The California General Public Utilities Code (GPUC) is typical of this type of code and establishes requirements for electric traction railroads and signaling systems. Governing jurisdictions may also identify unique requirements for high speed rail systems such as earthquake or wind/weather provisions.

If foreign technologies are selected, then the associated design criteria should be reviewed, adapted for the planned application and incorporated in the procurement documents. Other design criteria will describe system parameters, equipment, and component characteristics that define the technical features and resulting performance. It is beyond the scope of this Manual to specify the extent and detail of these requirements.

3.3.3 General Description

The system technology, route location and service characteristics should be evaluated to determine the applicability of Federal regulations. These will have significant impacts on the design standards and safety features that have to be incorporated. Examples are buff strength requirements, track standards and signaling standards, which are applicable to all US railroads that serve interchange traffic. An exclusive right-of-way dedicated to high speed passenger traffic to be operated within a single state or province, or by a specially constituted authority, may permit more latitude for selection of suitable design requirements. This evaluation will have significant implications for cost, safety and liability considerations borne by the operator, especially for train movements beyond exclusive right-of-way.

The sections below discuss aspects of high speed rail systems and vehicles and are not intended to be a design specification. They focus on high speed and very high speed rail as defined in Paragraph 3.5.1.2 of this Chapter of the Manual.

3.3.4 Performance

Performance is assumed to fall into a range of 110 mph (177 kph) or higher, within the specified speed ranges, and also includes acceleration, braking, and curving response. Active or passive suspension or tilt systems may be used but may not be essential to achieving or maintaining this level of performance.

3.3.4.1 Characteristics

Vehicle consists may be comprised of locomotive hauled individual cars, unit trainsets, or multiple unit configurations. Standard coupling or articulated trucks can be used, and axles/wheelsets are likely to have steerable or radial capabilities, provided via linkages or suspension components. Axle loads should not exceed acceptable high speed rail industry limits in current use. Disc brakes appear to be the most suitable for friction braking, however, dynamic braking should be a primary system with blending capabilities. The unsprung weight of the vehicle should be minimized. Propulsion systems can be electric traction power supplied by a catenary system or with interim on-board generation by gas turbine, diesel or other fossil fuel. However, a catenary based system is the most practical approach for the foreseeable future since the state-of-the art offers many performance, operational and economic advantages for future upgrading above 125 mph (201 kph).

3.3.4.2 Performance Requirements

Acceleration rates and brake rates should achieve the maximum levels permitted by passenger comfort and safety. Jerk limiting systems controlling external acceleration effects and wheel slip/slide control should be provided. Top speed is 110 mph (177 kph) or higher, with sufficient power for acceleration up to the top speed.

3.3.4.3 Operations and Controls

Load shedding control should be provided to limit current draw and maintain traction system voltage stability. Electro-pneumatic braking controls will maintain an even brake rate on all cars. The control console, ergonomic factors and cab layout will be established by the vehicle manufacturer in accordance with Federal regulations and should incorporate comment by vehicle operators. Operation at higher speeds has a significant effect on stopping distances, sight distances, power requirements, and control system response times. Operator and vehicle control topics are discussed in Section 3.5.

3.3.4.4 Compatibility

The traction power, signal and communications system, infrastructure, vehicles, terminals, facilities and operating practices should be internally compatible and be designed as parts of a system to ensure coordination of all elements and components, and the nature of the operation dictates compatibility issues. While it is preferable to provide high speed service on a dedicated right-of-way with specifically designed systems, other combinations of equipment and service are possible. However, the broader the range of service, equipment and travel speeds, the more complex the issue of compatibility becomes. Safety for passengers, employees, equipment and operations becomes an increasingly complex consideration as speeds and service mixtures are varied.

The design should also be responsive to owner/operator service and economic objectives. This could include number of tracks, station locations, track configuration, speeds, frequency of service, parking, stopping patterns, interlockings etc. Intermodal or other connecting transfer facilities must be suitable for each type of vehicle or system and adequate for the anticipated passenger volume and peaking characteristics.

3.3.4.5 Passenger Environment

The passenger environment is also addressed in sections relating to rail vehicles and passenger facilities. Passenger comfort, safety, amenities, and accessibility topics are detailed under specific requirements and headings. The primary consideration is provision of systems, vehicles and infrastructure to support and facilitate compliance with these requirements while enhancing the passenger environment. Passenger environment should be viewed from an all-inclusive perspective (portal-to-portal), from entrance to a system facility, whether by car, on public transportation or on foot, to departure from the system. The station and passenger facilities and services should equal or exceed airline standards of service and be commensurate with the service quality and cost. The passenger environment should remain relatively consistent with respect to temperature, humidity, and atmospheric pressure whether in a station, tunnel or on a moving or stopped vehicle.

Vehicle crashworthiness should be considered for both passengers and employees. This characteristic will influence the vehicle structural design, interior layout and choice of materials. Passenger cars should be joined with a fully enclosed weatherproof passageway between cars and all vehicles should be equipped with anti-climbers, collision posts, and appropriate draft gear. It may be useful to consider the presence of hazards to rolling stock along the right-of-way in the event of a derailment or train separation. Access for emergency vehicles onto and along the right-of-way should be incorporated in the right-of-way design. Route alignment and geometry will have a significant impact on passenger comfort because of its effect on speed and forces which the passenger experiences. Therefore coordination is essential between the civil aspects of the route and equipment behavior and tolerance for changes in route alignment, grade or curvature. Track maintenance standards should also be considered by designers since deviations from system designs will have a significant impact on passenger comfort, noise and ride quality, and wear and tear of the vehicle suspension and unsprung components. Designers of vehicles and right-of-way must consider the trip time and the duration of exposure to noise levels, centrifugal forces, vertical and lateral acceleration and jerk rates, and vibrations in relation to passenger comfort. The use of spiral transitions, superelevation and other features should contribute to maintaining passenger comfort levels. Additionally, the tolerance limits for right-of-way parameters should be easily maintained using standard maintenance equipment.

3.3.4.6 Environmental Impact

The operation of the vehicle, its systems, and facilities should be designed to prevent damage to the environment and protect against potential hazards. Such features could include containment or countermeasures for accidental release of undesirable fluids or substances, and strategically placed materials and equipment for response to emergencies. Any new project will be the subject of an environmental impact assessment. This assessment should include the offsetting benefits of the system if, for example, automobile and airline trips are reduced. Furthermore, the no-build impact on the environment needs to be analyzed, since it might result in increased auto travel, different commercial growth, or new airport construction. Construction of the system will normally be covered by applicable requirements to protect or restore adjacent disturbed sites, habitats and bodies of water, etc.

No substances or effects that are significantly harmful to human, animal or plant life should be produced or expelled. The potential effects of natural disasters such as earthquakes or extreme weather in causing secondary damage should not be overlooked. This should apply both within the confines of the vehicle and in its operating right-of-way and facilities and to employees, passengers and the general public.

Environmental considerations should include impacts on the air, water, and soil. Effects can be the result of normal operation or be caused by abnormal incidents and can include combustion, spillage, chemical reaction, heat, cold, wind, electrical conductivity or magnetic energy. Other less significant effects such as interference with radio, television (cable or airwave), telephone (hardwire or cellular) and utility services should be considered. The tendency for utility pipes, casing, and conductors which cross or run parallel to the route to conduct or attract electrical energy or provide a path for spilled fluids should not be overlooked.

If the trains are electrically powered via utility lines, substations and a traction power distribution system (e.g. catenary), all effluents and pollutants can be controlled at the point of power generation by the generating authority. This may be preferable and more effective than multiple local or individual controls. However, the transportation system owner should endeavor to incorporate regenerative braking technology to minimize the demand for energy. Operational practices or technologies to smooth out demand peaks should be encouraged. Renewable energy sources should be considered for power supply to remote field facilities or to supply battery chargers where feasible.

3.3.5 Safety and System Assurance

The primary objectives of safety and system assurance are to provide a high degree of protection and reliability and minimize downtime during maintenance and malfunctions. Quantitative goals/requirements should be specified for system elements where applicable. In addition to the quantitative goals to be met, requirements may be defined for a reliability program plan, specific analyses, prediction, and reliability demonstrations, depending on available field experience data for system elements from railroad industry sources.

3.3.5.1 Reliability Program and Submittals

Manufacturers of system equipment should establish, submit for approval, and maintain a Reliability Program and Plan, including:

- A detailed listing and description of each task;
- The timing of each task and related milestones;
- The organizational element responsible for each task;
- Identification of reliability problems requiring resolution;
- Procedures for recording reliability problem resolution.

Contractors for the Rail Vehicle, Train Control, Signaling, Traction Power and Communications Systems should be required to prepare Reliability Analyses and submit them for system coordination and approval.

3.3.5.2 Reliability Requirements

Contract documents should require the achievement and demonstration of reliability both by analysis and demonstration testing. Systems contractors should be required to develop and obtain approval of Reliability Demonstration Test Plans. New vehicle types should be subjected to a planned qualification test program at the Transportation Technology Center, Pueblo, Colorado or at an equivalent facility. Warranty provisions can be included in all civil and system contracts, to assure that costs of replacing and repairing defective materials and components are clearly the responsibility of the contractor. In addition to general warranties, which cover a time period from start up of operations, acceptance or delivery of a facility or piece of equipment, additional time warranties can be included in the vehicle contract. In addition to warranties included for specified time periods, additional warranty requirements relating to the maximum failure rates on particular components can be imposed.

3.3.5.3 System Maintainability

Each system element and its constituent equipment should be designed to permit ready access for maintenance. Maintenance personnel should have access for performance of maintenance functions, including failure location and isolation, disassembly and reassembly, repair/replacement as well as routine inspection/testing. Quantitative and qualitative maintainability goals should be specified for system elements where applicable.

(a) Maintainability Program

Manufacturers of the system equipment should be required to establish, submit for approval, and maintain a Maintainability Program and Plan, which should include:

- A detailed listing and description of each task;
- The timing of each task and related milestones;
- The organizational element responsible for each task;
- Identification of maintainability problems requiring resolution;
- Procedures for recording maintainability problem resolution.

Quantitative and qualitative maintainability requirements should be established, as appropriate, and should be incorporated into the appropriate contract documents. Maintainability should be analyzed during design, production, and testing of the equipment to evaluate the degree of achievement of maintainability design requirements.

(b) Maintenance Concepts

A detailed Maintenance Concept should be developed and submitted by each system equipment contractor for approval. The maintenance concept should include a description of how the contractor intends to achieve maintenance requirements identified in contract documents.

(c) Maintenance Analysis

A Maintenance Analysis should be developed and submitted by each system equipment contractor for approval. The analysis should describe all the maintenance tasks that operating authority personnel may be required to perform on the equipment.

(d) Maintenance Manuals

Maintenance department employees should be provided with detailed instructions that cover the servicing and repair of all system components. All suppliers and contractors should be required to submit maintenance manuals for approval that contain all the information needed to service, maintain, repair, inspect, adjust, troubleshoot, replace, and overhaul each component or subsystem.

(e) Training

Maintenance personnel should undergo a comprehensive training program for maintaining all system elements. The training should be sufficient for, and compatible with, system start up requirements, and should provide a level of education and ability to ensure the competent maintenance of the high speed rail system and associated equipment.

3.3.5.4 System Safety

There should be a management policy that safety be the primary consideration throughout the evolution of a system, from preliminary engineering through revenue operations. To fulfill the obligation of this policy, all applicable codes and regulations, technology and industry standards, should be used to ensure that each system achieves a level of safety that equals or exceeds that of the passenger railroad industry. Safety can be achieved during the preliminary engineering and final design phases by eliminating, minimizing, or controlling hazards through analysis, review and design selection. This includes provisions for emergencies such as an emergency communications network, on-site emergency equipment, access by emergency forces, and emergency preparedness planning in general.

(a) Safety Program

The objectives of the safety program should be overall elimination or control of hazards and assurance that no single point failure or undetected latent failure in combination with any additional failure would result in a hazard. A resulting acceptable level of risk and full compliance with FRA safety regulations should be achieved. The system should include health and safety provisions for maintenance and operational personnel that are equal to or exceed the requirements of state, province or regional Occupational Safety and Health regulations, and any Occupational Safety and Health Administration (OSHA), U.S. Department of Labor, and Americans with Disabilities Act (ADA) requirements.

(b) System Safety—Basis of Design Approach

Prior to preparation of specifications and design development, a Failure Modes Effects Analysis or Functional Hazard Analysis should be prepared to analyze the loss or malfunction of each system function and categorize its affect on the system, personnel, passengers and general public.

3.3.5.5 Human Factors

The objective of a human-factors program is to ensure compatibility between the physical and functional system design features and the human element during operation, maintenance and support of the system. This objective should be accomplished by applying human-engineering criteria to the design of the equipment. This effort is supported by design studies, operator-task analyses, maintenance-task analyses, and test programs to assess the inherent human-factors characteristics of the system. The primary measurement of human factors is expressed in terms of safety and cost (staffing) and system effectiveness (the influence on system availability, dependability, and capability).

The human-engineering characteristics of a system directly influence the mean time between maintenance (MTBM) and mean down time (MDT) quantitative factors of maintainability. As an example, if human error in operation of the system induces a failure, MTBM will be affected. Or, if maintenance tasks are difficult for the technician to perform, then MDT will be affected. In this regard, human-engineering and maintainability-design features combine to aid in establishing the operational-availability characteristics of the system. The two disciplines are also related in a number of other respects, as follows:

- Maintainability and maintenance-analysis reports form the baseline for generation of maintenance-task analyses;
- Manpower-requirement reports are considered in making maintenance policy decisions;

- Defined skill-level needs aid in establishing training requirements;
- Environmental needs are considered in facilities planning (lighting, heating, etc.);
- The content of maintenance instructions is made consistent with the abilities and needs of the system personnel;
- Maintainability and human-engineering design features reflect on one another.

Human-factors program requirements often result in generation of a system analysis, operator and maintenance-task analyses, personnel-requirements data, and training/instructional aids planning information. Certain aspects of such data analysis should parallel the maintenance analysis, specifically from the standpoint of facilities, maintenance tasks, personnel assignment and skill levels, and training requirements. In some instances, depending on organizational interests and available manpower, it may be feasible to cover these areas on a joint basis. In any event, the maintainability engineer should maintain full cognizance of available human-factors data when completing the maintenance analysis.

3.4 ROLLING STOCK

High speed rail systems have been under development in various parts of the world since the 1960's. These systems have evolved over the years to the greatest maturity in France, Germany and Japan, with more recent development in Sweden, Italy and Spain. The rolling stock for these systems have varying characteristics which are specific for operation in their respective countries. Each system has been designed to meet specific requirements including unique infrastructure and sub-system characteristics to which the rolling stock must conform. In addition, each rolling stock supplier has developed its own proprietary technology to satisfy those requirements.

North American conditions will present an additional set of design requirements as compared with the European and Japanese experience. Climatic conditions may be more severe, maintenance regimens may be less intensive, and regulatory requirements are likely to be more stringent. Lessons learned in the successful design of conventional North American rolling stock should be carefully applied in modifying existing proprietary designs for North American service.

This Section is concerned only with a brief review of the parameters required for a systems approach to high speed rolling stock design. Because its design must be undertaken as one part of a high speed rail system, it is most important to identify the specific interfaces of the rolling stock to other parts of the system.

3.4.1 General Description

The design of any high speed rail service must be undertaken from a systems standpoint, where the rolling stock is merely one of a number of major components of the total system. To this point, rolling stock, track, catenary, traction power supply, and signal and train control systems must be designed to function together to ensure successful operation.

Because of the proprietary nature of the various successful foreign high speed trains, this Section of the Manual will not attempt to address all of the various features of train design that must be accommodated, but rather will merely cite the necessary interfaces between the rolling stock and its operating environment. The manner in which each of these interfaces is addressed is worthy of detailed consideration in subsequent development of the entire system design; however, only a brief description of each interface is given in these guidelines.

3.4.2 Trucks and Suspension

High quality track, with very tight maintenance tolerances, has been developed for all of the successful high speed rail systems constructed to date. Equally important is the interface between the rolling stock and the track. Trucks, wheels, and suspension parameters should be addressed in a manner that allows them to function at their optimum as required by the track interface. A thorough investigation of track/train dynamics should be undertaken to assure that ride quality and safety of the vehicles are maintained. In particular, appropriate design tradeoffs are necessary in balancing high-speed stability requirements with truck curving and wheel load equalization requirements, and with operation on lower class tracks in terminal and yard areas.

Further in this regard, if high speed service utilizing existing rights-of-way is undertaken on an incremental basis, the use of tilt-body equipment may be desirable. Tilt-body vehicles must be carefully designed to be failsafe and to move the car bodies to offset the effects of high cant deficiencies in a smooth, safe and predictable manner while maintaining passenger comfort levels.

3.4.3 Current Collection and Primary Power System

High speed current collection by means of pantographs presents a serious design issue. Catenary/pantograph dynamic conditions must be carefully addressed to ensure the minimum interruption in contact continuity as well as to avoid undue stress and dynamic input to the flexible catenary system. Aerodynamics and power collection capability are extremely important aspects of pantograph design. Operation of a train with more than one pantograph raised at the same time has been found to excite the catenary excessively, thereby causing excessive loss of contact, especially if the pantographs are not at opposite ends of the train.

The development of the traction power supply must account for the power requirements of a number of trains operating on the same circuit at the same time. Regenerative braking can be used to augment the power supply system but care must be exercised to optimize line receptivity when incorporating this function.

3.4.4 Brakes

The ultimate safety of any high speed rail system will be dependent on the effectiveness and reliability of the vehicle braking system. Braking must match the characteristics of the right-of-way and the signal and control system, as well as provide comfortable deceleration. In most cases, this will require a primary dynamic brake, either regenerative or rheostatic, supplemented by friction braking with discs. Wheel tread friction braking, if used, should be minimized to preserve the integrity of the wheels and avoid overheating. Under some circumstances, foreign high speed rail networks have allowed the use of magnetic track brakes; this is an interface with the track system that should be carefully designed and considered before its implementation. Eddy current electric braking is also a possibility that is becoming more feasible with advances in the technology.

3.4.5 Train Control and Communication Systems

The train interface with the signal system is clearly of critical importance to operating safety, In this regard, stopping distances must be ensured for the worst-case braking conditions. In addition, the security and integrity of the signal system must be protected from the influence of electromagnetic interference and transients arising from the vehicles' propulsion systems. Many new signal systems utilize solid state devices that require special attention to preclude difficulties of this type, particularly with modern traction systems utilizing inverter drive propulsion.

3.4.6 Car Body Exterior

Other significant issues that must be addressed in the design of the rolling stock include environmental concerns such as noise and sound levels, both to the interior of the equipment as well as external wayside noise, which could disturb the public. Aerodynamic noise from car bodies and pantographs, as well as wheel/rail noise, may require special abatement procedures for very high speed operation.

3.4.7 Crashworthiness

In contrast to conventional North American rolling stock design, where car body buff strength has been a predominant concern, high speed rolling stock design focuses on maximizing the crashworthiness of vehicle car bodies through use of energy absorption techniques. Designs for crash energy management providing vehicle crush zones or zones of increasing strength have been applied. In addition, the interior of vehicles requires special treatment to preclude injuries from secondary collisions of passengers with other interior objects in the event of a serious accident. Other wayside interfaces dealing with safety are addressed in Section 3.5 of this Chapter.

3.4.8 Car Body Interior and Environment

A very important interface of the vehicles is with the passengers themselves. To this end, the design of the vehicles must provide for a good ride in a comfortable environment. Simple as this concept seems, there may be considerable difficulty in optimizing these two major features. Good ride quality is primarily dependent on the proper design of the suspension and car body tilting system (if used) and their interface to the track. But, in addition, the passenger seating must be properly addressed to optimize the interface with the passengers.

The HVAC system is another important passenger interface. Temperature and humidity control must be carefully addressed to ensure a comfortable environment for the specific climate conditions encountered by the service. The car body ventilation and fresh air provisions may require special attention. Depending on speed and certain wayside characteristics, special consideration may be necessary in the vehicle HVAC system design to accommodate the pressure changes of fast moving vehicles as they enter and leave tunnels or pass other trains or structures. Some foreign high speed vehicles have addressed this concern by sealing the train to preclude the development of rapid pressure changes inside the train. However if this is done, it should be addressed in the early stages of the vehicle design. In some cases, the design of the tunnel can be such as to avoid the development of pressures in excess of those acceptable to passengers and crew.

Finally, with respect to car body interior features, the Americans with Disability Act of 1990 (ADA) has imposed specific requirements on all rail equipment built for use in the United States. In this regard, there may be design features of foreign high speed trains that will not be acceptable in this country. Some of the more significant features to be addressed in the design of the car body include the placement of hand rails and provision of turning areas and clearances for wheelchairs and other mobility devices, particularly in doorways and aisles. The provision of wheelchair parking spaces and special facilities for their access to the cars, as well as to other features within the train such as good layout of service areas and toilets, is required.

3.5 Control Systems

For a set of applicable terminology and associated definitions refer to :

- *AAR Signal Manual* Section 1
- Appendix A—Glossary, *Safety of HSGT Systems, State-of-the-Art and Assessment of Safety Verification and Validation Methodologies*. Report # DOT-VNTSC-FRA-95-8.I.

3.5.1 General Definitions and Objective

The objective of this Section is to provide a general overview of signal control systems and to highlight the signal control system requirements for high and very high speed train operation. The regulations and recommendations for train control provided in 49 Code of Federal Regulations, Part 236, the AAR Signal Manual, or in expected future regulations provide a basis for all signal control systems.

The objective of a train control system is to control the movement of trains from point of origin to final destination in a safe and efficient manner. The operation of trains at very high speeds requires a control system that, at the minimum, continuously informs the operator of the allowable safe speed and that applies the train's brakes if the operator fails to comply.

3.5.1.1 Available Technology

The train control system can vary from a simple installation where information is made available to train operators via wayside signals (to assist them in the decision making process in the control of the movement of their trains) to a completely automated system in which manual involvement in the normal movement of the trains is unnecessary. The level of safety and efficiency achieved will depend on, among other things, personnel selection and training, human/equipment interface design, operational and emergency response procedures, and traffic density, in concert with the level of automation implemented.

3.5.1.2 Current/Proposed Requirements

The signal and control requirements are established based on the operating environment and speed range of the trains. Different signal and/or control systems can be used for different sections of the rail transportation network, as long as the maximum train speed in a section of the network is limited to be consistent with the signal and control system in effect and with the specific train's onboard control system. Refer to FRA Regulation 49 CFR Part 236, and to Safety of High Speed Guided Ground Transportation Systems—Collision Avoidance and Accident Survivability—Volume 4: Proposed Specifications—Part 3.6.2—Signal and Train Control, Report # DOT-VNTSC-FRA-93.2.IV, for additional information.

Wayside equipment capable of supporting moderate, high and very high speed operational ranges can be used to allow the control of trains in these three speed ranges as a function of the train's onboard control system. Thus, trains equipped to operate at moderate, high or very high speeds can be operated over the same territory depending on a complete risk analysis of the specific mix of freight, passenger and/or speed parameters. Obviously very high speed trains following closely behind moderate speed trains will not be able to take advantage of their total capabilities. In such an operation the maximum authorized speed limit will be restricted to the moderate speed range speed limit, unless the train control system positively determines that it is safe to permit correctly equipped and properly functioning high and very high speed trains to operate at higher speeds.

In general:

- Low speed range (0–59 mph {0–95 kph}) train movements are permitted under manual control with verbal instructions or train orders from an operations control center or dispatcher. All trains operating in territory that has no signaling system are restricted to this speed range regardless of their onboard train control capabilities.
- Moderate speed range (60–79 mph {97–127 kph}) train movements are permitted under manual control, using line side signals. Automatic means of detecting the position of the train on the track structure are used to activate restrictive line side signals for other trains. At junctions, interlocking systems prevent setting or changing switches or signals to a position that would permit conflicting train movements.
- High speed range (80–125 mph {129–201 kph}) train movements are permitted under the requirements stated for the moderate speed range except that a cab signal system is required but lineside signals are not required (operation from 80 to 110 mph {129–177 kph} is allowed with automatic cab signal, automatic train stop or automatic train control). This added system must provide cab signals, provide an audible warning whenever the cab signal indication changes to a more restrictive condition, remove propulsion and apply the brakes if the operator does not acknowledge the more restrictive cab signal indication within a preset time. An automatic train protection system as described for the very high speed range below is highly recommended for trains operating in the high speed range.
- Very high speed range (above 125 mph {201 kph}) train movements require an automatic train protection (ATP) system. The ATP system continuously compares the actual train speed

to the maximum permitted speed, taking into account speed limits for the individual train, temporary or permanent speed limits imposed because of track structure conditions, train control instructions and train braking capability. The ATP system must provide positive enforcement of all civil and operational speed limits. If the actual speed exceeds the permitted speed by more than 9 mph (15 kph), automatic propulsion removal and brake application must be initiated to reduce speed to a level at or below the permitted speed before manual operation can be resumed. The train operator must not be able to override the automatic removal of propulsion and brake application in any way that would allow the train to operate at a speed exceeding the maximum permitted safe speed by more than 9 mph (15 kph).

NOTE: These definitions do not supersede any established speed definitions existing in applicable operating or maintenance rule books, Federal or State regulations, guidelines or recommended practices. Thus for high speed and very high speed rail operations all trains must, as a minimum, be protected by a failsafe control system i.e., component or subsystem failures always result in the system reverting to a known safe state. The minimum system shall include failsafe route control and locking, in-cab display of current maximum safe operating speed, current actual speed and overspeed warning, and overspeed protection. If potentially unsafe situations, such as train overspeed, should develop and the operator does not take proper action, the system must automatically enforce safe operation.

3.5.1.3 Vital Subsystems

A minimum system requires vital subsystems incorporating system components that are highly reliable, have a minimum number of known failure modes, and have been designed as a system to ensure safe train movement. Vital subsystems are functionally responsible for train detection, speed limit determination, route interlocking, and overspeed protection (including actual speed measurement, overspeed determination, and train brake control). These subsystems must be highly reliable, fault tolerant (exhibit low down time) and ultimately be failsafe.

3.5.1.4 Nonvital Subsystems

There are a number of nonvital subsystems available for train control, which are not necessary for safe train control but are useful in improving the efficiency of the system's operation. They may include diagnostic functions, route control (automatic or remote), train speed regulation, automatic station stopping, scheduling, dispatching, management information (e.g., on time performance, operating statistics, etc.), communications systems, system status display (central control), etc. These subsystems are included to improve the train control system and to provide information to a central control operator for use in determining the best way to provide required services to the passengers. These systems do not need to be failsafe. They must be implemented in such a way that their functions (or lack of functioning) do not interfere with those of the vital subsystems.

3.5.2 Automatic Train Control (ATC) System

An ATC system may be comprised of Automatic Train Protection (ATP), Automatic Train Operation (ATO), and Automatic Train Supervision (ATS) functions. ATP provides protection for passengers, personnel and equipment from accidents due to unsafe train operations. ATO controls basic operations that would otherwise be performed by an operator and does so within the safe operating limits imposed by the ATP. ATS is the link between the central control operator and the system. It provides system status and provides means for the central control operator to monitor and initiate control requests for specific operations such as route alignment and schedule changes.

3.5.2.1 Automatic Train Protection (ATP)

The ATP function provides for safe operation. Safety shall be maintained under all circumstances, including any combinations of wayside power on or off, train power on or off, and all possible conditions of automatic operation. Under any of the above conditions or combinations of conditions where the ATP is inoperable or overridden, emergency and procedural means shall be invoked by operating personnel to assure safety of continuing operation.

The ATP system must provide protection to prevent personal injuries to passengers and personnel, and prevent physical damage to equipment or the appurtenant facilities within its area of control. Safety is of paramount importance. The system must be safe not only when all elements are operating normally as intended, but also when malfunctions occur. In order to maintain safe conditions when malfunctions occur, the system must be designed to be failsafe. Acceptable means of verification and validation of software, firmware and hardware must be employed for all vital systems and subsystems to assure their failsafe capabilities.

The ATP system includes as a minimum the vital subsystems listed in section 3.5.1.3. For high speed and very high speed passenger movements, the following provides a general description of their vital subsystem requirements.

- Train Detection—a train detection subsystem that determines the physical location of all trains is required. Any error between the actual train location and the detection subsystem's determination of its location, under normal operating characteristics or due to failure conditions, must result in safe train operation. This subsystem will interface with the subsystems used to determine safe maximum speed limits and safe route control.

  The train detection subsystem should provide broken rail detection as an integral part of its operation or a separate subsystem/detection methodology should be used to ensure that broken rails are identified and train protection against this hazard is achieved.

- Route Interlocking—the route interlocking system will interface with the train location and speed limit determination subsystems. It will ensure that train movements will be permitted only when they do not conflict with other train movements and are in accordance with the switch settings. This system will also ensure that switch settings and routes cannot be altered unless it is safe to do so by properly implementing the concepts of approach/time locking, route locking, and detector locking as needed. For further definition of these concepts refer to Section 3.5.4

- Speed Limit Determination—the determination of the maximum safe speed for a train at any point within the system is a function of the location and route of trains in its path, the status of switches in its path, the civil speed limits in its path (curves, stations, switches, etc.), the horizontal/vertical inclination of the tracks (grade affects the braking performance of the train), and the train's braking characteristics (including reaction times, worst case failure modes and safety factors). Therefore, this subsystem requires wayside (infrastructure) information, train location information and train characteristic information. In all cases (normal operation and for all failure modes) this subsystem must determine the actual maximum safe speed limit or a speed limit that is lower. This system must also have the ability to enforce temporary restricted maximum speed limits for track sections. These restrictions may be necessary due to track work or temporary conditions that make the normal maximum speed unacceptable.

- Overspeed Protection—the overspeed protection function should provide absolute speed enforcement, ensuring that speed of the train never exceeds the safe speed limit. The overspeed protection subsystem includes speed measuring devices that furnish signals that are a measure of the train's actual speed. If actual speed is below the maximum safe speed limit for the current section of track, then braking is not initiated by the subsystem.

  In a high speed system, if the actual speed exceeds the safe speed limit, the operator should be given an audible warning. If the operator fails to acknowledge within a preestablished time limit (1 to 3 seconds), the overspeed protection system should remove the signal that allows propulsion to be applied and holds the brakes off, causing propulsion to be removed and brakes to be applied. It should be possible either to reapply the brake hold off signal and allow the operator to resume control once the actual speed is below the maximum safe speed, or the system can be configured such that the train will be brought to a complete stop by the overspeed protection equipment and only then will it allow the operator to reset the system and resume control.

In a very high speed system, if actual speed exceeds the current safe speed limit, the ATP system should automatically remove the brake hold off signal which removes propulsion and applies the brakes. The vehicle operator cannot override the automatic brake application under any circumstances if the actual speed is greater than the speed limit.

3.5.2.2 Automatic Train Operation (ATO)

ATO functions may be included as options in the train control system to improve system efficiencies. These functions automatically perform operations normally completed by the operator in accordance with prescribed operating criteria but within the safety limits imposed by the ATP subsystem.

- Motion Control—the motion control portion of the ATO subsystem is responsible for starting, stopping and controlling the train's operating speed such that acceleration, deceleration and jerk are within acceptable passenger comfort levels and the maximum speed is below the safe speed limit established by the ATP subsystem.
- Station Stop—the station stop portion of the ATO subsystem is responsible for bringing a train to a controlled stop at the correct location within the station platform limits.
- Passenger Information Control—the passenger information control portion is capable of interfacing with equipment to provide audio and visual information to passengers. This information may be onboard the train and/or in station areas. The information to be displayed may include train route or destination, next station, arrival time, station being entered, train departing, train departure time, transfer information, special messages, etc. The exact messages to be displayed and the method of displaying these messages will be established by specific system application and the current applicable regulations (e.g., ADA). Audio messages may be in the form of prerecorded or preprogrammed messages triggered by the control system at the correct time.

3.5.2.3 Automatic Train Supervision (ATS)

The ATS subsystem monitors and assists in the management of the overall operation of the system. The ATS is not essential to continuing automatic system operations by ATO and ATP once they are initiated. ATS provides the link between the central control operator and the high speed rail system. The ATS system provides information to the operator describing the status of the tracks and trains on a real time basis. This information allows the operator to assess conditions throughout the system and to take appropriate actions. The operator may issue commands to initiate and terminate system operations, override selected automatic commands and perform other system management functions subject to the constraints imposed by the ATP system. No action or lack of action by the central control operator nor any malfunction of the ATS system may cause an unsafe condition or otherwise subvert or compromise the functions of the ATP system.

The system requirements during emergency situations should be considered in planning, design and implementation of the central control/ATS system. Depending on the role that the ATS system performs in the total system's operation, contingency plans, back-up operating procedures and/or back-up systems should be included.

Descriptions of all the possible ATS functions is beyond the scope of this section, but ATS may include functions such as: status and performance monitoring, performance control (reduce system operating speeds and acceleration rates to conserve power/fuel), train tracking, data logging, headway and schedule management, train routing, train dispatching, passenger information control, fire and security alarm monitoring, malfunction reporting, etc.

3.5.3 Control System—Propulsion System Electromagnetic Compatibility

Modern electric propulsion, electric generation and inverter equipment, especially that which employs solid state power switching devices, has been shown to generate energy in the frequency ranges used by signaling systems. Although the energy levels of these potentially interfering signals are a small percentage of the power handling capabilities of the generation equipment, they are significant when referenced to signaling equipment sensitivity levels. In the worst case, such electromagnetic interference may compromise the safety of signaling systems. Proper systems engineering practice, including frequency allocation and immunity filtering, should be applied to ensure system safety and compatibility.

3.5.4 Interlocking System

An interlocking system is employed to facilitate and safeguard the movement of trains at terminals and junction points. It is defined as: "An arrangement of signals and signal appliances so interconnected that their movements must succeed each other in proper sequence and for which interlocking rules are in effect. It may be operated manually or automatically." See the *AAR Signal Manual*, Part 1.1.1, 1991, Page 29. The interlocking system must protect against the track switch machine from unlocking and/or moving the track switch:

- When permission is being displayed (via cab signal or wayside signal) to allow a train to proceed over the switch;
- For a predetermined time after permission to proceed has been removed and a stop has been displayed with a train occupying the track section in approach of the switch; and,
- When a train is passing over the switch.

Route locking or signal indication locking is the part of the locking that prevents the changing of the switch position once permission to proceed has been displayed. Approach locking or time locking is the part of the locking that prevents the switch from being moved for a predetermined time after the permission to proceed has been replaced by a stop signal. The predetermined time is established to be long enough to ensure that any approaching train will have come to a complete stop (and not be within the interlocking) before the locking is released. Detector locking is the part of the locking that prevents the switch from being moved when a train is passing over the switch. The rules and recommendations for the application of interlockings is included in FRA regulations 49 CFR Part 236 Subpart C and in *AAR Signal Manual* Sections 2.2.10 and 2.2.12.

3.5.5 Communications

Several types of communication systems should be considered when installing or implementing a high speed rail system. Some are essential for operations and control, others can be added later to enhance passenger services or upgrade from a basic control system. If there is any thought given to enhanced passenger services, such as direct broadcast of radio or television programs to trains, then the communications infrastructure initially installed should have the basics for such additional communications implanted so that upgrading is easily done by adding terminals or modules without the need for a separate communications link.

3.5.5.1 Essential Communications

Digital and voice radio can connect the wayside stations to moving trains. For dispatcher to train crew communications, this can be a voice link or a digital voice system. A digital data link is needed for transmitting data between control centers and the train. The link from the moving train to the wayside is via radio, but the link from the control center to the nearest wayside radio station can be via fiber optic cable, microwave or even UHF or VHF base radio station segments. The fiber optic cable provides a good solution for high capacity including allowance for future expansion, and has immunity to electro-magnetic fields created by electrified lines.

Inductive communications technology is often used to transmit data from track to train via beacons, "wiggly wires," inductors or transponders. Even coded track circuits in the rails can transmit signaling data, speed commands, etc., to the train cab. Other systems that do not use coded track circuits can use digital radio links from the wayside to a train equipped with an on-board computer to handle the control function.

3.5.5.2 On-Board Communications

On board the train, voice radio may be advisable with portable handsets for crew member communications. Some suitable location on the train, such as a dining car or other mid-train position, should be equipped with a conductor's station including a 30 to 50 watt output radio to facilitate contact with the control center or local dispatcher. The conductor's station radio transceiver would be powerful enough to reach a wayside radio base station, which a handset normally could not reach because of its limited power (often less than 1 watt).

Additionally, a public address system incorporating a back-up power supply is essential on the train to keep passengers informed of the train's progress and to provide service and station announcements. This PA system often has a built-in intercom so that train crew members and service personnel can communicate with each other. Visual station announcement signs and other visual signs are required by new federal regulations for aiding disabled passengers.

3.5.5.3 Emergency Communications

During an emergency when the train is not operating under normal conditions, communications is of extreme necessity. Certainly, battery-powered handheld radio sets or intercom systems will be most useful, but standby power sources or batteries should be provided for the 30 to 50 watt radio to enable train crew members to contact a control center and local emergency services. There should be good, clear, prearranged interference-free, reliable communications to local fire and police departments along the high speed train route to ensure quick response to emergency situations by means of predetermined access points along the right-of-way. The radio communications load should be considered when sizing battery power for cars and locomotives.

From a communications standpoint, high speed rail will require a "smart" communications infrastructure to handle all the communications needs. Computers may play a key role in the communications facilities and can be provided to handle numerous functions.

3.5.5.4 Wayside and On-Board Defect Detection

Present communications links between trains and wayside detectors, such as hotbox, hot wheel, cracked or broken wheel and dragging equipment detectors should be used on high speed rail lines. Detector actuation by trains could provide voice/data transmission of detection results to engine and train crews. "No defect found" should also be transmitted so crews know it is safe to continue the journey. This information is usually also sent to a control center when a defect is found. Additionally, some railroads find it helpful for maintenance purposes for all hotbox detector readings to be sent to central control, especially when defects are alarmed.

Locomotive system status monitoring is an important feature of high speed motive power. Thus, data collected during operations could be downloaded via a radio link or via a hardwire connection after arrival at terminals. If problems develop enroute, alarms could be transmitted to a control center and to the locomotive engineer. Here again, a digital data link should have sufficient capacity to transmit data to and from trains.

3.5.5.5 Enhanced Passenger Communications

In addition to the communications systems previously discussed, enhanced passenger services may require additional facilities. A few are listed below:

- Public telephone service with phone booths at convenient locations or phone sets at each seat, which may need a separate train-to-wayside communications link.
- Radio or television reception would require a train-to-wayside interface; TV screens could be placed in seat backs for convenient viewing and radio earphone connections could be provided at each seat.
- Provision of a communications interface could permit the computer user to reach out to the "outside world" from the train. Also, electrical outlets could be provided at selected seats to conserve lap top computer battery power.
- Facsimile transmission to/from the high speed train could be considered, requiring a communications link and a facsimile machine.

As technologies in personal computing and telecommunications fields are subject to constant change and rapid evolution, it is important that vehicles and stations be designed to readily accommodate improved computer and communications systems. The basic railroad system designs should provide sufficient capacity (in terms of space, electrical power, conduit runs, etc.) to support technology retrofits that will naturally occur over their service lives.

3.5.5.6 Connecting with external services

An important aspect of communications for high speed rail is interface with public services, including facilities such as medical and police departments. Also important is communications capability for ticket orders and reservations, etc., including hotel and motels reservations and car rentals. High speed rail passengers could be provided with reservation facilities for a complete trip.

3.5.5.7 Communications: A Phased-In Process

The communications infrastructure need not be installed initially to get a high speed rail system up and running. It can be phased in, often on a modular basis, but one should consider what a complete system providing all the passenger enhancements might do to help market the service to the public. With this expanded system in mind, a basic system can be installed with enough capacity to handle added communications requirements without replacing the installed system. One should note that where high speed rail can be applied to an existing railroad, the basic system of communications for train operations might be in place, but it may not be cost-effective to retain such a system unless capacity enhancement is already provided.

3.5.6 Hazard Detection and Surveillance

This discussion outlines the various means of hazard detection and surveillance that have been employed on conventional rail systems as well as on existing high speed systems. The potential need for new or more elaborate devices to protect against hazards not previously addressed on existing systems is also noted. Specifically not included in this discussion are the signal and train control systems that provide the first line of safety for any rail system. It is assumed, however, that specialized detection and surveillance devices will utilize signal and control systems to annunciate hazards or to activate the preventive measures necessary to preclude accidents. In addition, no attempt is made to discuss highway grade crossing warning systems, and it is generally agreed that the presence of highway grade crossings is incompatible with high speed rail operations at speeds over 110 mph (177 kph).

3.5.6.1 Introduction

Hazards that may develop into potentially unsafe conditions are separated into two categories, based on their location: 1) on-board and 2) wayside. On-board hazards have been, until recently, most often detected by wayside sensors that "watch" the train as it passes certain locations. Recent developments with automatic sensors and microprocessor control systems permit the condition or health of vehicles to be monitored on-board the train and thus provide real-time continuous observation and timely detection of potentially hazardous conditions. Wayside hazards, however, are most practically detected by sensors at the most likely locations of the potential hazard. Hazard detection and surveillance systems may also be characterized by their placement in on-board or wayside locations. On-board detection systems are intended to detect hazardous conditions developing on the train, while various specialized wayside devices have been developed to detect both on-board (e.g., hotbox, dragging equipment) and wayside (e.g., floods, slides) faults or hazardous conditions. Note that train control systems typically provide broken rail detection through use of track circuits.

It is anticipated that developing North American high speed rail systems will operate at conventional speeds on certain segments of existing freight and passenger trackage to gain access to urban areas. At other times, the high speed trains may operate at higher speeds on routes shared with commuter, conventional passenger and freight trains. In either case, it is expected that a basic level of protection would be provided by conventional wayside detection devices and vehicle-borne detectors. High speed rail technology has proven to rank extremely high in terms of operational safety, and the risk of accident on routes devoted exclusively to high speed rail systems (operating on dedicated rights-of-way) may actually be lower than for other types of rail passenger service. But the risk of accident is dependent on traffic density among other things, and high speed rail equipment operating on the same track or shared rights-of-way with other types of traffic could experience an increase in that risk. In addition, the consequences or severity of accidents will clearly increase with speed.

The purpose of the wayside and vehicle-borne detectors on high speed rail systems is to further reduce the risk of accidents in recognition of the potential for an increase in their severity. Therefore, existing systems should be supplemented as necessary by additional devices designed to detect specific types of high speed vehicle failures. In addition, because of the potential for more serious accidents at high speed, the threshold for the application of detection systems may have to be lowered in certain cases. The details and operational functions of the various systems are not discussed herein, but rather a general description of the hazards and some of the usual and/or most likely means of detection is offered.

3.5.6.2 On-Board Detection and Surveillance Systems

At conventional speeds, the detection of some on-vehicle conditions can be accomplished by wayside detection, but as the speed of operation increases there is a need for more frequent monitoring, and continuous on-board detection may be necessary to protect against vehicle malfunctions and minimize human error.

(a) *Operator Alertness*

The most basic operator alertness ("dead-man") device applies the brakes if a spring-loaded pedal or handle is released. It can be easily defeated by the operator. More recent use of movement or function detecting devices are less problematic, but still can be defeated so that they fail to detect abnormal operation or unsafe conditions. Improvement of the reliability and certainty of function of these devices is necessary to ensure they are impossible to defeat. In most cases, complete speed supervision is warranted including control of both traffic responsive (signal) and civil speed restrictions.

(b) *Hot Journal*

Although some high speed systems have found wayside detection adequate for their level of journal inspection and maintenance, wayside detection may be less suitable for high speed operation because it is intermittent. Continuous on-board detection of each journal bearing with temperature sensors, with wayside detection systems as a support system may be more desirable. The on-board system may have one or two trip levels—"warning" and "danger"—with the indication transmitted to the train's operating compartment where the operator can take the action prescribed by rules. It may also be appropriate to monitor the rate of change of bearing temperature. Local car odor or visual indicators will not be suitable.

(c) *Hot Wheel*

No existing on-board system has been used for this hazard, which would most commonly be associated with tread brakes. Some extension of the hot journal detection system or application of a second similar system would appear possible. Another possibility is the development of a stuck brake detector for both disc and tread friction brake systems.

(d) *Derailment Detection*

Development of systems that sense the degradation of ride quality or truck performance has been suggested as an index to incipient derailment. Ride quality detectors that monitor vertical and lateral acceleration are sometimes used to establish maintenance cycles for trucks and wheels and to ensure that truck hunting is detected quickly. Other possible parameters for detection include broken or deflated springs, and differences in wheel speed. Such devices have been subject of experiments but some have not yet been fully or reliably developed for derailment detection applications.

(e) *Pantograph Condition*

Although not yet in use or deemed necessary in North America, some means of sensing the potential failure of a high speed train's pantographs may become necessary to avoid a total failure. The pantograph/catenary interface is very delicate and failures are usually catastrophic to the overhead catenary system and can also cause significant damage to vehicles.

(f) *Doors*

Passenger carrying vehicles must have a door control system that prevents train movement if doors are open while the train is stopped and that stops train movement if doors become unlocked or open while it is moving slowly. Such protection is usually provided by "zero-speed" detection and "propulsion-inhibit" circuitry along with sensors to detect the position and lock condition of each individual door. Door control circuits and mechanisms must make door opening impossible above "zero" (+ some tolerance) speed.

Door control systems should be equipped with obstruction detection and automatic re-opening features to protect passengers against entrapment if any doors are prevented from fully closing. Systems should also be equipped with by-pass functions to permit train movement under prescribed rules in the event of a sensor failure or other fault in the door safety circuitry.

Emergency door operation must be possible without operating power so that entry/egress can be made in emergency situations. Apparatus for emergency manual door operation should be clearly marked and provided on both the interior and exterior of vehicles, together with instructions as to how and when emergency door opening is to be initiated. The entire door system should be subject to a detailed safety analysis.

(g) *Pressure Sealing*

There is a possible need for on-vehicle detectors to sense the degradation of internal vehicle pressure if the trains are sealed or carry positive pressure in a manner similar to aircraft cabins. This is a matter of both safety and comfort for passengers and crew with regard to the effects of pressure waves generated by passing trains and operation through tunnels.

(h) *Fire*

Protection against fire is a key element of system safety, and fire in a moving train is a major hazard to passengers, made worse by high speed. Vehicle-borne fire and smoke detectors should be provided to provide prompt annunciation of unsafe conditions to passengers and personnel, since safety is of prime importance. In addition, dedicated fire suppression systems with automatic initiation should be considered in areas where high temperatures or high voltages are normally present in locomotives, without requirement for any crew involvement.

Fire detectors and suppression devices should also be provided in other appropriate areas and controlled as determined by a comprehensive system safety analysis. In particular, careful consideration should be given to how and where conditions are annunciated to operating personnel, to the protocols for activation of fire suppression systems, and to procedures for passenger evacuation including avoidance of train-stops at locations where evacuation may not be feasible. The HVAC systems in affected and adjoining vehicles should be immediately shut down to prevent mixing of smoke and fumes with fresh air.

All vehicles should be built utilizing fire-resistant materials, which will not produce toxic fumes or smoke, and should have readily accessible fire extinguishers.

3.5.6.3 Wayside Detection and Surveillance Devices

A number of wayside hazard detection devices have been developed in various parts of the world for both high speed and conventional passenger, and freight routes. These devices are not used universally but have been developed for and applied to detect specific types of hazards.

(a) *Broken Rail or Guideway Integrity*

Both conventional and audio-frequency railway track circuits provide detection of broken rail conditions through the signal system. The track circuit is interrupted when the circuit is opened by a break in a rail. Both types of track circuits have limitations in their ability to detect broken rail conditions due to current leakage through ballast, joint bars, tie plates, incomplete breaks and similar circumstances.

(b) *Hot Journal*

For conventional North American railroad practice, protection against hot journals and the hazards of derailment due to journal failure is provided by wayside "hotbox" detectors. Detector spacing is determined by individual railroad policy and the speed of operation. Spacing is typically on the order of 15 to 20 miles (24 to 32 km). Detector placement is intended to protect interlockings, bridges, and other high-value facilities.

The use of wayside "hotbox" detectors on high speed rail routes can be considered to supplement the protection provided by on-board hot journal systems, particularly near stations, interlockings, and in other limited speed territory. It is expected that complete reliance on wayside "hotbox" systems for high speed routes will not be economically or technically feasible, due to the high density of installations required to provide a suitable level of protection as well as potential difficulty with the speed of detection.

(c) *Wheel Impact Load Detector*

Eccentric wheels or flat spots can cause impact loads that threaten the integrity of concrete ties and other track material in addition to accelerating the fatigue of bearings, axles and other truck components. Ride quality and wayside and interior noise levels are also adversely affected. Wheel Impact Load Detectors ("WILD") utilize wayside sensor installations to detect impacts (above a predetermined threshold) to the track for defects such as out-of-round wheels. This type of detector has been installed by railroads using concrete ties or slab track construction at locations such as tunnels or bridges. Such devices would be useful for screening rolling stock prior to its entry into high speed territory.

(d) *Dragging Equipment*

Dragging equipment detectors have heretofore been placed at wayside locations to detect vehicle-borne apparatus or parts that have become disengaged and hang below the level of the top of rail.

These devices are placed on the track in advance of interlockings or switches where equipment dragging from the train could pose the threat of derailment. These devices should continue in use on high speed systems but should be more closely spaced to provide more frequent sampling.

(e) *Over Dimension (High-and-Wide)*

Many railroads have developed reliable and generally simple photoelectric cell-activated high-and-wide detectors for freight routes to detect lading or car equipment that has shifted out of the clearance envelope. While such devices would not be generally necessary for high speed rail services on routes where the train make-ups are of known and consistent size and cross section, it is possible that devices of this type could be used at junction points to protect high speed routes from over-dimension cars and lading which might at times use them as detours from the normal freight routing, and at other selected locations where the high speed right-of-way is shared with freight trains. The clearances of the high speed route might be more prohibitive in these cases than on the normal dedicated route or shared rights-of-way.

(f) *Earthslide*

Slide detection fences have been used historically to protect the right-of-way in areas where avalanche, rock fall or other loose earth hazards could encroach on the right-of-way. Such fences are usually connected into the signal system in a manner that will drop all approaching signals to "stop" on either side of the condition when debris falls against the fence and causes the activation of electrical contacts.

(g) *Washouts*

Areas subjected to frequent or historic conditions of high or rapidly moving water, such as flash floods, may be provided with detection devices to sense the rise or velocity of water in or near culverts which may be inadequate to carry the total storm run-off. The right-of-way may be washed away or flooded in such cases and significant damage could occur. When such conditions are imminent, wash out protection devices would provide sufficient warning to stop all nearby trains.

(h) *High Water*

Similar in nature to washout protection is the high water detection device, which detects the rate-of-rise and absolute level of water around particularly vulnerable locations such as bridge piers, embankments, levees and culverts, which could face inundation by rapid storm run-off conditions. Conventional rain gauges could also be utilized to provide early warning of impending high water conditions. Some railroad operators routinely utilize private weather forecasting services to provide real-time notification of localized but severe weather conditions which might affect operations.

(i) *Earthquake*

Seismometers have been used in Japan to detect earth movement and shocks near the Shinkansen lines. These are site-specific applications of special seismometers, which are connected into the signal system and the central traffic control office to provide early detection of earth movement and shock in order to stop trains as soon as such problems are noted. Freight railroads in the western U.S. also monitor earthquake-sensitive locations, although not so directly, by utilizing the services of professional seismology laboratories to keep them advised of earthquake related conditions.

(j) *Wind*

Anemometers to measure wind velocity and direction could be set to trigger when conditions reach pre-determined levels considered hazardous to the movement of trains or to the catenary. Conditions or areas where local vegetation or debris accumulations could be blown into the right-of-way should also be considered for such devices.

(k) *Snow*

In the northern sections of the Japanese Shinkansen lines, snow detectors have been installed to annunciate the onset of snow fall and to measure its rate of fall which, along with prevailing and forecast temperature and wind conditions, can be used to predict the accumulation. In such cases, the operation of automatic or remotely controlled snow removal apparatus (i.e. water sprays and heaters for melting snow) can be initiated at all track switches and other appropriate locations.

(l) *Bridge and Guideway Alignment*

Protection of bridges and guideway is most important, particularly where high water or earth movement is possible, or where structures are vulnerable to collision from highway vehicles or marine shipping. The development of optical devices to verify proper alignment of selected critical guideway members could be useful in providing for early detection of misalignment.

(m) *Catenary Tension and Position*

The application of a catenary tension detection system is suggested to alert a control center or to provide a local indication of a degradation of catenary tension. A system to detect or sense physical displacement beyond allowable limits would seem to be technically feasible with doppler or optical techniques. Such conditions may occur through either structural failure or by the catenary's failure to properly adjust to temperature changes. Catenary tension and position, together with pantograph performance, is one of the most delicate of the vehicle-to-wayside interfaces.

(n) *Fire*

Smoke or fire detectors are most frequently used in congested areas or on wooden trestles, snow sheds, bridges or near other track-side structures where train movement would likely be endangered by fire. These detectors take the form of smoke, heat, and/or toxic fume sensors which may be used to alert authorities or control signals, fans, or sprinkler systems, etc. The need for use of fire detection devices is site specific. Logical candidates for high speed rail applications include station areas, electrical equipment rooms, control facilities and other areas where passengers, personnel or high value equipment are present.

(o) *Train Presence or Movement Detection*

The use of presence detectors is usually associated with track circuits and the signal system to detect the occupation of tracks in specific areas. Movement detectors may also be used to show motion and direction of movement.

(p) *Tunnel Protection*

Particular attention should be given to tunnels, where the hazards of derailment or accident are amplified by the confined quarters and the impeded access for evacuation. Response to concerns of this type requires well lighted walkways and the proper ventilation of the tunnel throughout its length, the detection of abnormal pressure conditions and particular attention to the potential for abnormal temperature, smoke, fire and fumes. In addition, of course, there is a need to detect tunnel lining material which may loosen and fall to the track from the sidewalls or ceiling. Further, dependent on climatic conditions, it may be necessary to monitor moisture and temperature conditions to predict the formation of stalactites, stalagmites and other ice formations. Attention to some of these hazards in tunnel design can minimize their risk. Various types of detection devices would be needed to provide these types of protection, however, all are within the realm of existing technology.

(q) *Intrusion Detection*

In addition to the normal hazards expected through the operation of trains and the general degradation and wear and tear of the right-of-way, a high speed rail system is also more vulnerable to intrusion into its right-of-way by vehicles, other foreign objects, and persons, and must be more carefully protected. Particular consideration should be given to fencing - especially in densely populated areas and perhaps for the entire length of the right-of-way. Guideway fencing will also protect against wildlife, trespassers, and vandalism. Many existing high speed rights-of-way are walled or fenced for their entire length to prohibit intrusion of this type. In addition such fencing will at least deter the migration—either deliberate or accidental—of other foreign material such as rock fragments and discarded residential or industrial items onto the right-of-way. Such fencing might also be equipped with a means to detect tampering or climbing, which should be connected to the central control office that would be responsible for initiating appropriate action.

Areas of particular concern for intrusion to the high speed rail right-of-way are locations where the high speed rail line passes under a highway or other type of structure. For the French TGV system in such areas, it has become necessary to provide special vertical fencing or barriers (depending on the level of risk) to prevent automobiles and debris from dropping or being thrown by vandals to the track from the bridges. At some locations, horizontal fencing is installed over the catenary and track and interlocked with the train control system to detect intrusion. Special intrusion attention of this type could be confined to specific vulnerable areas of the high speed rail route (cuts and tunnels etc.). The adjacent tracks of other carriers should also be considered for high speed systems utilizing common rights-of-way with other networks. However, it is difficult to predict where hazards of this type might take place and, recognizing the likely consequences of such intrusion, it may be prudent to fence or otherwise protect the entire right-of-way.

Occasionally it may be necessary to provide crosswalks across the high speed right-of-way for maintenance workers and other staff. In such cases there will be a need for some sort of signal system for pedestrians to permit safe crossings. The TGV line has such a system which annunciates the approach of trains with a red crossing signal light when there is less than 30 seconds of time to negotiate the crossing.

(r) *Grade Crossings*

For the purposes of this Section, it is recommended that highway grade crossings in high speed territory be completely eliminated. However, where highway grade crossings cannot be eliminated and in densely populated areas where the high speed trains may be traveling at more conventional speeds, special attention will be necessary. Discussion of grade crossing safety issues is beyond the scope of this Section.

3.6 ELECTRIC TRACTION POWER

The information and guidance provided with respect to Electric Traction Power aspects of High Speed Rail Systems is supplementary to information given in Chapter 33, Electrical Energy Utilization, of the AREA Manual. A section cross reference is given in each instance where the existing guidelines are relevant also to high speed rail applications.

3.6.1 General

When high speed rail service is electrified, electric traction power is comprised of power supply from utility companies to substations with power distribution by means of an overhead catenary system. High speed rail systems may be dedicated to high speed train services, which then requires that the system design of electric traction power be totally specific to those services.

3.6.2 Traction Power System

The general outline of the power supply system is similar to that of other electrified railroads, with transmission lines feeding into dedicated substations, usually railroad-owned, as discussed in Chapter 33 of this Manual.

3.6.2.1 Lines to be Wired

All tracks of electrified high speed routes should be electrified for train operation purposes. Where high speed trains run on other routes for access to passenger terminals, to maintenance facilities or for through running purposes, only assigned tracks and alternates need be considered for electrification. Route/track diagrams can be used to verify the choice of operating scenarios with all affected departmental personnel of the railroads involved.

3.6.2.2 Performance to be Achieved

The electric traction power demand for high speed trains is considerably higher than that for other types of passenger trains, due to the higher speed and to the longer sustained acceleration required. It is recommended that all or most substations have two transformers, providing sufficient capacity in reserve to enable all conceivable power demands to be met with only one transformer in service. The use of dual transformers, dual power supply sources and dual busbars provides a high level of redundancy and ensures that the probability of total loss of supply is extremely small.

3.6.2.3 Traction Power Systems

In addition to reliable operation and performance of substation equipment discussed above, it is important that the utility company power supply maintain a very high level of reliability. There should be two separate sources of incoming power supply, preferably direct from major high voltage supply grid circuits with proven high availability ratings.

The safe electrical operation of the traction power system requires safety standards for insulator creepage paths, air clearances and electrical switching arrangements. The switching and feeding sub-assemblies should provide for fault detection and protection relay operation, including zone relays, that can distinguish between heavy load currents and fault currents, and can attain virtually instantaneous disconnection of electrical supply when a fault condition occurs. Full information and details are provided in Chapter 33 of this Manual.

3.6.2.4 Sectioning Diagram

The full extent of the catenary system and associated substation, switching station and paralleling station locations should be shown on an electrical sectioning diagram. The catenary system should include sufficient sectioning to facilitate isolation of electrical faults to individually controlled sections of the operational track system. Starting with the normal feeding configuration, a number of disconnect switches are used to energize all catenary sections with all normal and alternate feeding scenarios.

Some disconnect switches may need to be included for abnormal situations such as the temporary loss of power supply from a substation or main feeding point, or the inadvertent stranding of a train at a phase break. Disconnect switches for these situations are normally open and would only be closed when the abnormal situation occurs, after appropriate adjustment from the normal arrangements. Additional information concerning the principles and normal sectionalizing practices recommended for use are provided in Chapter 33 of this Manual.

3.6.2.5 Substation Feeding and Supply Locations

The selection of substation locations is an iterative process involving a number of factors which sometimes conflict with each other. The nature of the high speed rail route or network and the situation of routine high power demand zones creates natural locations for preferred siting of substations. The high reliability requirements for utility power supply sources will usually restrict the number of alternative locations to a few choices. The availability of environmentally acceptable substation sites may further limit those choices, especially in urban or scenic areas. A provisional scheme for substation siting should be developed and verification procedures should be applied by implementing a series of computer studies. More than one scheme may need to be studied in difficult situations, involving the alternative types of feeding systems available for use, and providing alternative solutions for management review. Further information and details of recommended procedures are provided in Chapter 33 of this Manual.

3.6.3 Catenary System—General Description

A catenary system provides the means of distribution of electrical power to moving railroad trains, specifically in this instance to high speed passenger trains. All new catenary systems consist of a constant tensioned contact wire supported by one or more messenger wires which are in turn supported by brackets or cross span assemblies on adjacent poles. Some older catenary systems have variable tensions with fixed wire deadends. Detailed descriptions, definitions, standards and concepts relating to catenary systems are provided in Chapter 33, Part 4.1, of this Manual.

3.6.3.1 Car Clearance Gauge

See Chapter 33, Part 2, Clearances for information and guidelines on locomotive and car clearance gauges, subject to potential superelevation, sway, tilt and bounce allowances relevant to high speed rail operations.

3.6.3.2 Electrical and Mechanical Clearances

See Chapter 33, Part 2, Clearances and Part 4.2, Catenary Systems Design Criteria for information and guidelines on electrical and mechanical clearances. There are no significant additional clearance requirements for high speed rail operations, but a greater amount of pantograph uplift at supports may require provision of increased allowances for pantograph passing clearance past support hardware assemblies.

3.6.3.3 Contact Wire Height

The range of contact wire height is normally controlled by the minimum height of overhead bridges and structures, together with the maximum height of contact wire necessary to comply with high wire clearance requirements at highway grade crossings or other facilities. The pantograph performance requirements for high speed rail include a design preference for small, low-mass pantographs and a catenary design need for a small vertical range of contact wire height.

High speed train operations over 110 mph (177 kph) should preclude the retention of highway grade crossings, which virtually eliminates the need for contact wire heights greater than a nominal normal height. This normal height is determined by the addition of wire sag allowances and tolerances to minimum wire heights, plus allowances for long term track ballast rise and maintenance tolerances. The normal wire heights may need to be increased where right-of-way is shared with freight service for local access or through operation.

3.6.3.4 Dynamic Interaction and Compliance

The dynamic interaction that occurs between a catenary system and train pantographs traveling along the catenary is a complex relationship. The operating requirement is that the pantograph provide a continuous flow of electrical current to the locomotive for traction and auxiliary purposes, at all speeds and under the most adverse vehicle and track tolerances. This is best accomplished without significant arcing or loss of contact, which in turn is achieved by maintaining a reasonably uniform contact pressure between the pantograph and the contact wire. Minimum contact pressure for acceptable performance would typically be in the range of 8 to 12 pounds (36 to 53 Newtons).

A proven dynamic simulation should be developed to confirm the compatibility of the selected pantograph and catenary system scenario before undertaking any final design activities. The simulation should verify that upward pressure applied by the pantograph on the contact wire, including dynamic increases due to high speed, will remain within an acceptable range for current collection purposes.

High contact pressures cause progressively greater contact wire uplift as a pantograph passes a given point. This can cause excessive mechanical wear of pantograph carbon wear strips and the contact wire, especially near catenary supports. The static pantograph contact pressure used in North America has traditionally been in the range of 22 to 28 pounds (98 to 125 Newtons), especially where high electrical current levels are involved. The increasing U.S. application of European and Japanese practice and introduction of modern low mass pantographs has included use of lower static pressures in the range of 16 to 18 pounds (71 to 80 Newtons).

The contact wire rise and fall also manifests itself as a traveling wave, which moves forward at a speed dependent upon the wire's natural frequency, which in turn is a function of the wire weight and the tension applied. A basic principle of catenary design is that this wave speed should always be greater than any potential train speed, to avoid any prospect of a pantograph "traveling along" with its own standing wave. This situation becomes more complex with multiple pantographs on a train, especially if they are relatively close together, due to the interactions involved, especially at speeds over 150 mph (241 kph).

For high speed rail applications, the inertial effects of pantograph reactions to catenary features, contact wire gradients, and track irregularities become so significant that the following guidelines are applied to catenary design procedures:

- Contact wire tension—should be kept as high as possible, using present day contact wire materials, pole spacing, blow off considerations and strength of support hardware, involving preferred use of constant tension systems;
- Contact wire gradients—should be at least 5 times train speed in miles per hour, doubled at transition spans; for example, 1 in 750 gradients are used for 150 mph (241 kph) operation;
- Catenary system—should have maximum feasible uniformity of compliance (vertical stiffness) in order to minimize the fluctuation of dynamic pantograph response;
- Catenary configuration should be designed to accommodate the maximum pantograph pressure developed at the highest speed, including aerodynamic uplift.

The following guidelines should be considered during selection or design of pantographs:

- Pantograph location—only one per train if at all feasible; otherwise two are used, placed at opposite ends of the train preferably at least 1,000 feet (305 m) apart. If more than two are necessary, then mini pantographs should be used with much smaller variation in contact wire height, since multiple pantographs operating together suffer mutual degradation of their current collection performance, especially at higher speeds;
- Pantograph head—should have a minimum feasible dynamic mass consistent with mechanical strength and frangible design requirements;
- Pantograph pressure—tends to increase in proportion to the square of the speed. It should be kept as constant as possible at all speeds, in both directions; often assisted by use of aerofoils;
- Malfunction and damage prevention—the potential damage that can be caused to a catenary system by a malfunctioning pantograph before the high speed train can be stopped is so great that the pantograph head should incorporate an automatic dropping device and the frame should include a weak-point break-away feature.

3.6.3.5 Safety Considerations

The principal information regarding the safety of Electric Traction Power for High Speed Rail is given in Chapter 33. The use of high level platforms may generate additional electrical clearance safety considerations and compliance with the National Electrical Safety Code or equivalent National Code is required.

All overpasses and other overhead structures on electrified high speed rail routes will require the use of safety barriers in all areas of feasible pedestrian access. Barriers should be at least 6 feet 6 inches (1.98 meters) high, should extend at least 10 feet (3.05 meters) longitudinally from the nearest catenary, and should include features preventing any side access around the end of a barrier. The material usually selected is rigid aluminum sheeting, which should not include any protrusions providing feasible footholds for climbing access.

Grounded safety screens, barriers and anticlimbing guards should be included in the design of signals and other railway facilities and features within 10 feet of a catenary system, and the use of fencing or safety cages should be considered to prevent vandalism or unauthorized access when required by specific circumstances.

3.6.3.6 Electrical Characteristics

The electrical characteristics for High Speed Rail typically include sufficient power supply system capacity for sustained acceleration from station stops or from in-route delays up to the high operating speeds required. The electrical current demand for each train is substantially higher than for other types of passenger trains, due to the high drag coefficient at high speed and the current drawn by on-board facilities. The use of motive power utilizing regenerative power capability may also be considered, subject to load limitation, traffic levels and potential for future growth, over-voltage prevention and harmonic content limitations.

3.6.3.7 Grounding and Bonding

The requirements for Grounding and Bonding are generally in accordance with information given in Chapter 33 of the Manual. The grounding system must prevent any risk of an unsafe condition occurring either within the electric traction system, at stations, or in the immediate surroundings along the right of way. The electric traction and signaling systems both require the use of appropriate bonding connections, circuits or other devices at project-specific spacings to ensure the failsafe operations of trains and supporting infrastructure. Compliance with step-and-touch protection standards should be given high priority in all areas accessible to passengers and other non-railroad personnel.

3.6.4 Catenary Safety Design

There are few standards, codes or specifications that directly refer to catenary systems for North American applications other than the guidelines provided in Chapter 33 of the Manual. The catenary industry applies local engineering and construction procedures to their relevant extent, together with appropriate input from overseas railroads. This composite design basis should be monitored to verify that all technical, operational and environmental safety requirements are adequately provided for, to the satisfaction of the appropriate regulatory organization. The following are some of the key requirements to be considered when an overhead electric catenary system is required for a high speed rail project.

3.6.4.1 Conductor Parameters

The conductors for an overhead catenary system consist normally of a contact wire supported by hangers from one or more messenger wires. The conductor tensions at each operating and non-operating limiting condition have to be reviewed to verify that adequate safety factors are provided. The recommended minimum safety factors for high speed rail projects are 2.5 on messenger wire tensions and 2.0 on contact wire tensions, relative to ultimate conductor breaking loads throughout the lifespan of the wire. Supplementary factors to be considered are the wear allowance for the contact wire, typically 25 percent or less for high speed rail projects, and an annealing allowance in respect of gradual reduction of ultimate conductor breaking loads.

3.6.4.2 Technical Design Data

The safety requirements applicable to the various types of support systems, poles, foundations, portal structures, cantilevers and related items such as contact wire wear are detailed in Chapter 33, Part 4, of this Manual, and the only additional need arising for high speed rail projects concerns the use of larger catenary pole offset discussed in Paragraph 3.6.4.5.

3.6.4.3 System Features and Arrangements

The operational performance requirements for high speed rail projects make it essential to incorporate some form of automatic tensioning into the catenary system. This feature compensates for temperature changes and maintains a constant high mechanical tension in the catenary conductors to facilitate optimum current collection performance. The need for automatic tensioning creates a number of special system features and arrangements. These include catenary overlaps between individual tension lengths of catenary, typically spaced as needed at up to a maximum of about one mile (1.6 km), dependent upon the system design selected.

The system design has to provide for safe pantograph passage at high speeds through such system features, including those at high speed crossovers, while also providing for dynamic uplift and lateral sway of the pantograph and the train. The pantograph horns provide an additional safety feature for pantograph passage through catenary system features and arrangements, especially in view of the significantly greater pantograph uplift experienced at very high speeds.

Some form of overrun protection should be provided wherever passenger trains are routed past facing connections with diverging non-electrified track. This will prevent safety and operational problems in the event of a raised pantograph being inadvertently routed onto the diverging track, with resultant potential teardown of overcrossing feeder, static, communications or signaling cables and wires. Overrun protection can be provided by adding short sections of catenary to guide diverging pantographs beyond overcrossing obstructions, or by raising or rerouting such obstructions beyond the maximum pantograph reach and sway.

3.6.4.4 System Sub-Assemblies

The automatic tensioning system has to cater for temperatures ranging from minimum ambient to maximum operating temperature, which includes provision for solar heating and peak levels of electrical load heating. The pulley/balance weight tensioning assemblies (or spring tensioners used where space is very restricted) can be fitted with inertial stops to hold the tensioning assemblies if sudden breakage of conductor occurs. Temperature stops can be provided to operate below a defined wire temperature, typically 20 degrees F. (–7 degrees C.) to restrict the amount of catenary system sag that would otherwise occur when heavy icing conditions are encountered, and to reduce the need for severe speed restrictions at such times.

3.6.4.5 Poles and Foundations

The safety requirements for high speed rail projects include those which apply to other types of railway electrification activity, as discussed in Chapter 33, Part 4 of this Manual. An additional consideration concerns the minimum and normal pole clearances from adjacent tracks. Where feasible, especially when applied to new construction, the normal clearance should be at least 14 feet (4.27 m) to provide for passage of a high speed train with one or more derailed vehicles which is in the process of making an emergency stop, or experiencing a progressive derailment. Trackside features including catenary poles and foundations can be set back sufficiently to permit such passage without sideswipe contact and increased risk of injury to passengers and crew. The choice of pole design can include consideration of likely impact scenarios involving high speed trains to minimize the danger to passengers and crew. Features to be evaluated should include the style, ductility and break-away elements of poles considered for use.

3.6.5 Catenary System Installation Design

The catenary system selection and design process should consider how installation of the selected system can be undertaken in accordance with safe working practices and in compliance with OSHA and local regulatory requirements.

3.6.5.1 Installation Procedures

The design process should take full advantage of feedback from prior construction experience, and should avoid construction staging configurations that involve partly constructed system supports, conductors or other features which are structurally weak or incapable of withstanding severe weather conditions. If construction work has to be undertaken on an existing operating railroad, the catenary installation plan should be conceptually developed as a number of individual work stages, most requiring relatively brief on-track time and incorporating a maximum amount of advance sub-assembly work under more controllable conditions at a construction depot.

3.6.5.2 Shop Drawings

Wherever detailed design of material is part of a procurement contract scope of work, or where proprietary materials are being supplied, the contractor should be required to submit detailed shop drawings. These should illustrate the design, function, means of manufacture and specification appropriate to the specific item, and should give reference to the originating conceptual design or assembly drawing.

3.6.6 Catenary Installation Procedures

Installation of high speed rail catenary systems follows the normal practices of the catenary industry, as discussed in Chapter 33 of this Manual. However, many catenary design features are much more restrictive when designed for high speed rail projects and most installation tolerances are considerably less than customary for other types of electrified railroad construction.

Specific installation procedures should be developed during the detail design process for each construction phase, with the ultimate objective of providing a high performance level of traction current collection by high speed trains.

3.6.6.1 Foundations

The interfaces between ground conditions and catenary pole foundations or footings are less crucial than most other interfaces since the catenary support assemblies mounted upon the poles generally include adjustability in all dimensions. Most foundations rely on side bearing loading capacity and the installation procedures should ensure that excavation is accomplished without significant disturbance of the hole sides, roadbed or ballast compaction. If precast foundations are used, the tamping material used to set the foundations in excavated holes should be capable of filling all spaces and voids to avoid foundation movement or rotation when catenary system loads are applied. Further information on foundations is given in Chapter 33, Paragraph 4.2.8 of this Manual.

3.6.6.2 Poles

There are two mounting styles used for catenary poles, being either a plain section pole directly implanted into a cored foundation, or a bolted base pole mounted directly on top of a foundation or footing. Poles should be properly aligned in conformance with primary catenary system loadings. The pole setting process should incorporate backward unloaded rake of catenary poles when necessary to compensate for maximum loaded pole deflection under normal environmental conditions. The cross bridge of portal structures should have an upward camber while unloaded before erection to compensate for vertical weight and catenary system-induced loading deflection to prevent the cross bridge sagging below a horizontal configuration. Further information on catenary poles is given in Chapter 33, Paragraph 4.2.7 of this Manual.

3.6.6.3 On-Site Measurements and Site Fabrication

The key interface between installed catenary poles and adjacent tracks is established when pole installation is completed. The as-built pole settings from centerline of track should be measured, and information on proposed track realignment and final superelevation should be verified. Where critical, the vertical dimensions between high rail level, top of pole, and any fixed connection points should also be measured. These measurements should be dated and recorded as the data base for all subsequent stages of the catenary system installation.

Some on-site fabrication of catenary supports can be undertaken, especially to accommodate last minute changes during the catenary installation process, but the over-riding priority should be to fabricate assemblies or sub-assemblies at off-site locations with better equipped and environmentally controlled facilities. When some on-site fabrication becomes necessary, any cutting of support tubes, conductor strands, or other materials should be carried out in accordance with approved supplier instructions, including application of paint or other protective treatments to all raw surfaces or cut ends of ferrous items.

Splicing of conductor strands or other materials should only be undertaken with prior knowledge and approval of the system operator, and should be in accordance with pre-approved supplier splicing procedures.

3.6.6.4 Brackets, Support Assemblies and Dead-ends

The as-built pole settings discussed in Paragraph 3.6.6.3 should be used to verify fabrication sizes of fixed cantilever brackets prior to installation on site. This check should be carried out prior to fabrication where dimensions and space requirements are particularly critical.

The single track hinged cantilever and deadend brackets to be mounted on support poles alongside or between tracks should be pre-assembled using standard structural sections and hardware on a purpose made jig at the construction base, using the as-built pole settings and superelevation data. They would then be transported to site and erected by use of a highway boom truck or similar rail mounted vehicle. Further information is provided in Chapter 33, Section 4.1.4 of this Manual.

3.6.6.5 Catenary System Conductors

All support brackets and dead-end arrangements have to be completed and in place before the installation of catenary system conductors can be started. The process should begin with temporary attachment of running out pulleys for the messenger wire, and continue by anchoring the messenger wire at a dead-end and running out the wire at a sufficient steady tension to keep it raised clear of all obstructions, trains and other on-track and trackside equipment. Care should be taken, particularly on curves, to keep the wire close to its final height and lateral position. The wire is temporarily over-tensioned, typically overnight or longer and by at least 5 percent, to take up most of its early life cycle creep before setting it at a nominal unloaded tension.

Preassembled hangers are then mounted on the messenger wire at predetermined positions, including a temporary wire clip which can be used for instant contact wire attachment. A similar running out technique is used for the contact wire, taking care to avoid creating any indentations, vertical kinks or rotational twists during the running out process. Supplier or site splicing of catenary conductors is normally not accepted and damaged wire-runs should be replaced in their entirety on a new installation project.

The completed conductor system should be set to its final tensions and all hangers and support clamps should be set as required in their proper vertical and lateral positions.

3.6.6.6 Section Insulators and In-Span Materials

The catenary system is divided into electrical sections by insertion of section insulators and phase breaks at pre-determined locations, as discussed in Chapter 33, Section 4.1.5 of this Manual.

Depending upon the design selected, it will normally be necessary to install these items where the catenary is close to or coincident and parallel with the pantograph centerline at the elevated contact wire level. Other special purpose in-span items should be installed at this time, including all continuity jumper and feeding connections, and great care should be taken to optimize the contact wire alignment and profile.

3.6.7 Catenary Testing

The testing of a catenary system installation for a high speed rail system has to consider the higher degree of accuracy, compliance with tighter installation tolerances and the electrical current collection performance standards that have to be achieved. The testing procedures should be developed as a series of cumulative steps which together should result in a fully adjusted catenary system ready for high speed service.

It should be understood that satisfactory high speed current collection performance is also dependent upon well aligned and surfaced tracks, fully operative vehicle suspensions, and properly functioning vehicle pantograph assemblies.

3.6.7.1 Pantograph Inspection

As installation and adjustment of the catenary system is completed for each tension length and feeding section, a dummy pantograph mounted upon a trolley or on a mobile framework should be used to check contact wire and messenger wire positions at support and midspan locations. The transition zones at overlaps and interlockings should be checked to verify that vehicle pantographs achieve a smooth transfer between parallel or overcrossing contact wires, making due allowance for pantograph uplift and sway expected at the specific location.

3.6.7.2 Height and Stagger Inspection

All contact wire heights and staggers (offsets) at supports, together with wire heights and offsets at mid-span between supports, should be measured and recorded. The data would be initially used as a cross reference with the design to check for anomolies; subsequently, the data would become a permanent as-built record for use by maintenance forces or during upgrade or modification of the installed system.

3.6.7.3 Electrical Testing

The installed catenary system should be visually checked to verify that all insulators are in place and are unbroken and that all feeding and continuity jumper connections are complete and do not bridge electrical sectionalizing arrangements. The placement of bonding and grounding connections should be verified and all temporary grounding protection connections applied during installation are removed.

The electric utility supply to electrification substations should then be energized and a program of electrical testing initiated to verify function and operation of all electrical equipment items, including step-down transformers, circuit breakers and disconnects, relays and metering devices, supervisory control and related circuits, and various types of backup systems including standby batteries and alarm circuits. Particular attention should be paid to timely equipment operation for clearance of simulated faults at various locations.

The catenary system should be energized, section by section, and correct function of lineside equipment including disconnects, auxiliary power step-down transformers and phase break transponder units is verified. The no-load voltage levels at substations and at extremities of long feeding sections should be recorded, and clearance of simulated catenary faults verified. The presence and magnitude of stray currents in wayside signal and communication circuits is verified for compliance with applicable standards; further testing should occur in collaboration with slow speed and high speed train operational testing.

3.6.7.4 Slow Speed Testing

A program of slow speed pantograph testing should be initiated as electrical testing is completed for each area of the catenary system, typically for 20 to 30 track miles (32 to 48 track kilometers) of a project. This slow speed testing can be undertaken either with a self-propelled, purpose-built test train equipped with a live or grounded pantograph, or a standard locomotive or trainset suitable for slow speed running. If feasible, the head of the pantograph should be calibrated with a scale so that the amount of any excess movement of the contact wire towards or onto the angled horns can be noted.

The effectiveness of this slow speed testing will be further improved if the test train operates along every feasible route through an electrified area, including emergency moves which may not ordinarily be permitted by the signal and train control system. When possible, a video monitor and recorder should be used to record inspection findings, including some means of correlation between catenary location, track mileage and overhead obstructions.

Particular attention should be directed to electrical and mechanical clearances at supports, and to alignment and level of incoming contact wires as they come into the operational riding area on top of the pantograph head.

3.6.7.5 High Speed Testing

A series of high speed train test runs should be undertaken when construction of sufficient areas of catenary system has successfully complied with electrical and slow speed testing procedures and requirements, and all necessary adjustments have been made. A large amount of data may be recorded during high speed test runs, particularly with regard to train equipment function and operation. It should be verified that passage of the pantograph along the contact wire complies with pantograph performance requirements under all operating conditions. The pantograph mechanical performance should be monitored, particularly with regard to fluctuation of pantograph contact pressure and general avoidance of any loss of contact during normal operation. Pantograph operation should be monitored through areas with strong, prevailing side winds and across special purpose catenary assemblies such as section insulators, phase breaks, moveable bridges, minimum clearance overhead bridges and through overlaps.

3.6.8 Catenary Maintenance Manuals and Procedures

Overhead catenary systems are typically designed to have a service life of at least 30 years, ranging up to 50 years in many instances. Systems are intended to be maintenance free, but the typical harsh railroad environment requires that effective preventive maintenance procedures be adopted. The use of specialized custom-built hardware also requires provision of supplier maintenance manuals, specifying necessary routine inspection activities.

3.6.8.1 System Maintenance

An overhead catenary system and its component items should be subjected to a number of prescribed periodic inspections to verify system functions and to check for component wear, misadjustments or faults.

Correct functioning of automatic constant tensioning systems is of particular importance, and routine checks of termination assemblies should be carried out at regular intervals.

A dedicated maintenance team should be established at a strategic location, equipped with purpose-built hi-rail and wire train vehicles sufficient to carry out both routine maintenance and emergency repair tasks.

3.6.8.2 Maintenance Standards

Maintenance standards for electric traction systems should either be established as part of the design and installation basis, or be included in supplier documentation packages approved during procurement of equipment, hardware assemblies or components. Many maintenance standards will be identical or similar to construction acceptance standards and tolerances; a few may include additional tolerance or performance nonconformance provisions not available to the original supplier or installation contractor.

The maintenance standards for catenary systems on high speed train routes require close conformance with contact wire heights and gradients to avoid high vertical acceleration of pantograph head assemblies, high contact forces or momentary loss of electrical supply to the locomotive. Some maintenance standards involve two or more levels of tolerance or other conformance parameters, which facilitate continuing operation on a degraded basis, that could include lower acceleration rates or lower maximum speeds, similar to track classification standards.

3.6.8.3 Periodic Tests

Part of the preventive maintenance program recommended for high speed rail routes should be a series of performance tests applied both to the power supply equipment and to the overhead catenary power distribution system. Power supply equipment testing can routinely include one or more operations of circuit breakers and disconnects which have no recorded operations within a defined period, typically one year.

Catenary system performance testing can routinely include one or more pantograph operation video recording along each main track of the high speed route. One video should be taken at or close to the maximum operating speeds along the entire route. If any degradation of pantograph performance is identified, a second video should be taken through route sections of interest or concern, with train speeds controlled to match the design balancing speeds through each curve. Direct comparison and joint evaluation of the two videos should help to establish the cause of any performance degradation, which typically could be related to track misadjustment, vehicle suspension misadjustment, pantograph frame joint deficiencies, catenary system misadjustment or any combination of deficiencies.

A permanent maintenance history log should be maintained for each power supply facility and for each electrical catenary section. This log could highlight any long term recurrence of deficient conditions, and help to develop corrective actions for implementation prior to system or equipment failure occurrences.

3.6.8.4 Safety during Electrical Fault

Electrical faults are caused by one or more factors, including inadvertent faults caused by wild animals and birds, insulation failures in power supply and catenary systems, or equipment failures on board locomotives or other motive power vehicles. All interconnected, energized equipment and systems have to be fully bonded throughout to prevent any possibility of floating electrical potential remaining in conductors, wires or equipment components when the traction power system is de-energized.

All non-energized metallic and other electrical conductive items including support poles and frames, lineside fencing, station metalwork and all other such items within the traction power system electrical zone of influence should be effectively grounded. Metal components should be bonded together and all assemblies and isolated items should be connected to a ground wire system, a ground grid or to grounding rods, in accordance with NESC requirements and electrification industry practice.

Circuit breaker and relay systems installed at power supply substations and switching stations provide protection against electrical faults. Each circuit breaker provides connection of electrical power from an energized busbar to an individual region or track-specific section of the catenary system. The occurrence of an electrical fault would be detected by relays which cause the circuit breaker to open within a few milliseconds. Since many faults are transient in nature, most circuit breakers should be set to reclose automatically within a few seconds to restore power to the section. If the fault persists, the circuit breaker will lock open until the fault is investigated and repaired. Special distance zone type relays should be used to differentiate between high traction power loads and actual faults.

The circuit breakers for catenary sections at yards, station platforms and other areas where passengers or railroad personnel are routinely present should not reclose onto a fault, but instead should lock out immediately for on-site investigation of the fault cause and affects.

3.6.8.5 Safety during Switching Operations

The switching of electrical power is carried out in accordance with prescribed procedures by authorized personnel at an electrification system control center that would usually control an entire route or region. The procedures used should include some form of interlock or logic data-based system to prevent simultaneous supply of power to a section from more than one source, phase or substation, particularly when alternate feed arrangements have to be implemented while a fault is being investigated and rectified.

Safety measures adopted include siting of all switching equipment in high security situations, compounds or buildings, with intrusion detection systems connected to the control center for all facilities which are normally unattended.

3.6.8.6 Safety during Maintenance Operations

Maintenance of traction power supply and catenary system facilities should be preplanned for implementation during light traffic periods or during short overnight system shutdowns. All access and work activities should be in accordance with prescribed procedures that prevent any approach within specified limitations until all power has been disconnected and permanent or temporary grounds have been installed at either or both sides of the access area. Any circuit breakers or disconnects that could be inadvertently closed and reenergize the area of access should be locked open and tagged by the person responsible. The tags should not be removed until all personnel have been accounted for as being outside the specified access limitations and maintenance grounds have been removed or disconnected, and the tags should only be removed by the person who initially placed them. The remote operation of circuit breakers and disconnects may require supplementary safety procedures to achieve safety of personnel and equipment.

Safety during specific maintenance activity requires that all personnel should clearly understand and acknowledge the limits of temporary access, and should be fully trained, regularly certified and checked in respect of their knowledge and familiarity with maintenance procedures and activities being undertaken.

3.6.8.7 Safety during Abnormal Environmental Conditions

Some high speed rail routes may run through regions subject to occasional abnormal environmental conditions, requiring special parameters in the system design basis or special operating procedures. The overhead catenary system design basis may need to include provision for abnormal conditions including heavy ice on catenary wires and occurrence of very high wind conditions. To ensure the safety of passengers and equipment, train speeds should be limited when wind speeds exceed some specified amount, typically above 55 to 60 mph (89 to 97 kph). Furthermore, train operation should be suspended if wind speeds reach hurricane levels of 75 mph (121 kph).

Other abnormal environmental conditions can affect the integrity of traction power systems, and these were discussed in paragraph 3.5.6.3 of this chapter.

3.6.8.8 Security of Isolator Switches

The overhead catenary power distribution system incorporates numerous disconnect isolator switches, usually mounted on trackside catenary poles or structures. Most disconnects should be motorized to facilitate operation by remote control, but all disconnect isolator switches should incorporate some form of manual control for local or emergency operation. The security of these disconnects is maintained by use of removable handles and special locks to prevent unauthorized disconnect switch operation. Furthermore, other security measures should be included in the prescribed procedures which control the operation of all control equipment items including isolator switches.

3.6.9 Catenary System Tools and Equipment

A number of catenary system components and assemblies make use of nonstandard fixing and assembly techniques requiring the use of special purpose tools, jigs and equipment. These are usually provided by the relevant hardware suppliers as part of procurement packages, and are retained with maintenance departmental tools and equipment stocks.

Specialized equipment as used by the installation contractor provides effective access to the catenary system location above tracks, including hi-rail equipped bucket trucks and platform trucks. Extensive high speed rail systems benefit from provision of rail vehicles for catenary wiring and inspection, equipped with flat roof working platforms and dummy working pantographs. These give an effective means of undertaking routine inspection and rectification work, and facilitate rapid replacement of worn or damaged contact wire.

3.6.10 Environment

High speed rail operations have to interact with a number of environmental requirements and conditions. These include conditions that occur naturally, restrictions related to safety and operability, and man-made alterations to the natural environment.

3.6.10.1 Physical Environment

The physical environment that a high speed train travels through is partly dependent on the region traversed between the route end points, which in turn is influenced by local decisions concerning routing choices. High speed trains usually have widely spaced intermediate stops, which provides flexibility for evaluation of alternative routes when avoidance of undesirable physical environmental conditions is a major factor. Typical undesirable conditions related to traction power and catenary system installations can include unstable land strata, recurrent heavy snow and icing conditions, and high levels of isoceraunic activity (lightning frequency and intensity).

3.6.10.2 Meteorological Basis

The seasonal and recurrent weather conditions along a high speed rail route have to be established for use as the basis of design. Some parameters are provided in NESC and other codes and guidelines; icing levels are stipulated to be heavy (0.5 inch/13 mm radial ice), light (0.25 inch/6mm radial ice) or not applicable, according to the region or locality involved. More information is provided in Chapter 33 of this Manual.

3.6.10.3 Electrical Interference

Individual elements of traction power and catenary system installations have to co-exist with the electrical interference environment that is present. To a large extent, this environment is self-generated by current flow and voltage present in major system elements including catenary system wires and feeders. Other electrical interference may also be present, usually from parallel high voltage electrical transmission lines.

Other systems in the vicinity of high speed rail routes also have to co-exist with electrical interference caused by the presence and operation of the railroad traction power facilities. Systems affected can include low voltage power circuits and various kinds of communications, signalling and train control circuits, particularly when they run parallel to the route for significant distances. A number of interference mitigation measures and procedures are available for use when interference levels become excessive or create safety concerns. Typical circuits that may be affected include local telephone and video cable circuits installed along parallel residential streets.

Electrical interference does not affect fiber optic cable systems since no electrical conductor is present in the longitudinal cable system.

3.6.10.4 Pollution/Contamination

Past experience on railroad electrification projects included the need for mitigation of industrial air pollution and contamination. Most harmful forms of air pollution have now been eliminated, but some benign contamination still exists in unique local circumstances. Such contamination may occur naturally and its presence should be recognized by incorporation of appropriate design features, treatments or materials. One typical example is the presence of high salt concentrations in moist on-shore winds at route sections along stretches of sea or ocean coastline.

Committee 18 has submitted an introduction to a new chapter on Light Density and Short Line Railways for publication in this Manual.

Chapter 18—Light Density and Short Line Railways

Scope

The material in this chapter is supplementary guidance for railroad lines categorized as light density or short lines. While these categories have no exact limits, for engineering purposes they are generally defined as follows:

Light Density

A railroad line carrying less than 5 million gross tons of traffic per year. (Note: 1 million gross tons approximately equals the passage of 7,600 loaded 100-ton cars, or an average of 21 of these cars each day of the year.)

Short Line

A railroad not large enough to economically justify the use of large scale production maintenance techniques or to have fully staffed engineering and maintenance of way departments. Maintenance of way staff is sized for routine maintenance activities and may not perform this work full time. Engineering and design work, and perhaps much of the track and structure maintenance, is typically done by contract.

Proposed 1996 Manual Revisions to Chapter 27—Maintenance-of-Way Work Equipment

Part 1—General

Page 27-1-1. Replace the current Article with the following:

RECOMMENDED COLORS FOR PAINTING MOTOR CARS, ROADWAY MACHINES, WORK EQUIPMENT AND RAIL GUIDE WHEEL EQUIPMENT

The most suitable colors for painting Motor Cars, Roadway Machines, Work Equipment, and Rail Guide Wheel Equipment are as follows:

Equipment	Color	Standard Spec. No.
Motor Cars	Yellow	13538
Roadway Equipment	Yellow	13538
Roadway Work Equipment (cranes, etc.)	Yellow	13538
Work Equipment (Jordan Spreaders, etc.)	Mineral Red	10076
Rail Guide Wheel Equipment	Black	17038

The predominant colors of equipment will be a specified color, but some parts or components may require custom painting (reference AREA Committee 27—Specifications for On-Track Roadway Equipment, Section 17—Paint). Some of the more notable items are listed below:

Component Area	Color	Standard Spec. No.
Gasoline Tank	Red	11086
Diesel Fuel Tank	Green	14110
Hydraulic Reservoir	Blue	15180
Coolant Reservoir	Gray	16251
Lifting Lugs	Black	17038
Safety Locks	Red	11086

The colors listed under Standard Specification Numbers are consistent with those indicated in the "Federal Standard 595-B, Colors Used in Government Procurement."

NOTE: It should be understood that some Railroads may have special painting requirements which will supersede the above recommended practices or guidelines.

Page 27-1-2. Replace the current Article on Care and Maintenance of Maintenance of Way Equipment with the following:

GENERAL CARE AND MAINTENANCE OF MAINTENANCE-OF-WAY EQUIPMENT (INCLUDING RAIL GEAR EQUIPMENT)

The varying conditions on different railroads do not permit the universal acceptance, in all details, of any specific outline or an organization for the use, maintenance, and repair of the equipment. To determine the proper organization best adapted to produce maximum service and productivity, at a minimum cost, from its fleet of equipment each railroad must review and analyze various conditions which will greatly influence these factors. Some of the more important areas are listed below.

1. The quantity and age of the major and minor units of equipment.
2. The level of mechanization and sophistication of the equipment.
3. The type of managerial organization established for utilizing, supervising, and maintaining the equipment.
4. Geographic location and total trackage involved with the railroad.
5. Organization of Equipment Repair Shops, i.e., centralized and decentralized, Engineering, Mechanical, or Fleet jurisdiction, manpower, etc.

However, there are certain principles in the use and maintenance of machinery which will be found desirable, if not essential, if the equipment is to be maintained and used economically. Special attention should be directed to the following areas:

1. The organization for supervising and maintaining the equipment should be headed by a practical railroad employee, with sufficient executive ability and solid mechanical knowledge to supervise the maintenance and operation of all equipment on the system. The appropriate department should have authority, and sufficient personnel to institute and enforce regulations for the maintenance and operation of the equipment. Duties should embrace direct or indirect control of mechanical details of the equipment, in both field and shop, the supervision of maintainers, involvement with necessary reports and managing data base information. He/she should work closely with the supervisory forces in the assignment and use of the equipment. He/she must also work very closely with the Purchasing and Material Department as it relates to acquisition of new equipment, component parts and the distribution of supplies required for the maintenance and repair of equipment.
2. It is desirable that adequate instructions for the care and operation of the equipment be issued. Such instructions, developed by the AREA, were first published in the proceedings in 1947 and were later reprinted in handbook form under the title "Manual of Instructions for Care and Operation of Maintenance-of-Way Work Equipment." This manual is presently in the process of being revised.
3. Cooperation on the part of supervisory officers in seeing the instructions are carried out are of the utmost importance. This cannot be secured unless each such officer recognizes the importance of the work he/she is supervising in an effort to keep the equipment in operation. In order to achieve maximum results of equipment productivity and availability, through proper operation and preventive maintenance procedures and practices, the local and upper management levels must give their full support to the program.
4. Prevention of disabling conditions in any machine is as truly maintenance as is the correction of such conditions after they have developed; and prevention will save the loss of time and money required for repairs.
5. As a result of following a thorough preventive maintenance program, the need for emergency and/or unscheduled repairs can be substantially reduced. A preventive maintenance program is basically a repair prior to failure philosophy and can be characterized by the following:
   a) Keeping equipment clean.
   b) Operating the equipment safely and within its limits of capacity.
   c) Well controlled lubrication program.
   d) Performing required inspection, adjustment, and scheduled maintenance at the proper intervals.
   e) Utilizing a formal record keeping system i.e., operators and maintenance personnel, daily log, machine history file, failure reporting system, cost involved with equipment

use, which will provide a solid foundation for a computerized record keeping and cost capturing system.

f) Ongoing training program for operator and service/maintenance personnel.

6. Material Department operations should be properly located and adequately stocked so that repair parts, when needed, can be supplied with the least delay. This area can be streamlined with the use of highway type van trailers. properly sized, well organized, and adequately stocked to eliminate some of the current problems associated with using rolling stock tool cars. Some of these problems are delay in timely spotting of the car at the work site and damages to the parts inventory due to rough handling. The use of relief equipment and power plants will reduce delays in the event of equipment failures.
7. In scheduling the shopping of equipment for major repairs, consideration should be given to the capacity of the shop as well as the conditions of the equipment. In so far as possible, the equipment used in seasonal work should be overhauled during the slack season.
8. Shops for the repair of equipment should be centrally located and under the control of the appropriate department. These shops should be equipped with the necessary tools, shop machinery, and in general provide repairs in the most timely and cost effective method.
9. The expense of maintaining the equipment can be reduced through the adoption by each railroad, of the fewest number of makes and type of the equipment required to meet its needs. Such a restriction to adopt its standards will reduce investment in stock parts and lower maintenance costs.
10. Adequate reports and records should be prepared as a means of maintaining close check on the use being made of the equipment, the care of it, and to assist in passing judgment on purchase of new equipment.

Part 2—Roadway Machines

Page 27-2-31. Add the following new section on Rail Guide Wheel Equipment specifications:

SPECIFICATIONS FOR RAIL GUIDE WHEEL EQUIPMENT

Specifications must be reviewed by the manufacturer and returned to purchaser with bid. Compliance with each section must be acknowledged by checking “yes” or “no” at the close of each section; remarks may be made there or on a separate attachment.

1. GENERAL

1.1 Scope

These specifications cover rail guide wheel equipment.

1.2 Model

Equipment shall be of the latest type in production at the time of delivery. Components which are obsolete, nearing the end of production, or out of production shall not be used.

1.3 Reliability

Design, construction, and materials used in the equipment shall assure that it will function reliably and efficiently in sustained operation under hard usage in an adverse railway environment, including but not limited to severe grades and superelevation.

1.4 Workmanship

Equipment shall be free from defects such as incomplete welds, welds that cross welds, corrosion, loose or improper fastenings, leaks or contamination, and any other defects that could impair its operation or serviceability.

1.5 Maintenance

Design shall provide for ease of service, replacement, and adjustment of components, including filters and fluids (if applicable), with minimum disturbance of other elements.

1.6 Legal

Machines shall comply with Federal and State environmental, safety, and health regulations in force at the time of delivery. In the event of conflict or variation between regulations and these specifications, the most restrictive requirement will apply.

Comply: Yes ______ No ______ Remarks: ________________________________

2. MATERIAL

2.1 All steel plates, shapes, bars, and sheets shall be of a quality that has good weldability, high impact resistance and high notch toughness at low temperatures (0 degrees Fahrenheit to –40 degrees Fahrenheit). Steel items shall be of alloy and grades normally used for maintenance of way equipment and railway rolling stock. Design of structural members subject to normal working loads shall have a minimum design factor of 2 to 1. If structural members are subjected to impact stresses, a minimum design factor of 3 to 1 shall be utilized. It is generally recognized this is only a minimum recommended guideline and increased design factors may be required as necessary.

2.2 All fasteners shall meet strength requirements of ASTM A449 or stronger. All bolted applications shall have at least two full threads protruding beyond the nut after the fastener has been properly torqued.

2.3 All non-ferrous metals shall be of alloys having strength and corrosion resistance suitable for the service intended.

Comply: Yes ______ No ______ Remarks: ________________________________

3. BRAKES

3.1 Self-propelled machines capable of speed in excess of 10 M.P.H. on straight and level track shall be equipped with spring applied, power released brakes on all wheels. Service brakes shall be progressive in force application and capable of sliding all wheels on dry, sanded rail when fully applied at maximum travel speed.

3.2 Brake shoes shall not be applied so as to cause a bending force in an axle. A single shoe at the top of a wheel may apply downward pressure along a line passing through the wheel center within 15 degrees of vertical. Double shoes applying equal opposing forces may be used in any position.

3.3 In emergency situations a system must be provided for releasing each brake in not more than 2 1/2 minutes per wheel.

3.4 Air brake systems must utilize SAE J1402 Table A (formerly Type E), DOT #FMV-SS 106-74 Type A1 air brake hose with reusable fittings. Brake system must have pressure regulator, pressure gauge and standard truck type reservoir.

Reservoir must conform with SAE J10-B specifications. Manufacturers shall also be able to provide these types of reservoirs which meet ASME specifications for certification when requested as an option. System must be activated by standard truck type, foot or hand operated, control valve and one truck type, quick release valve.

3.5 Air brake system must maintain operating pressure, above 70 psi but not more than 105 psi. An audible low pressure alarm or warning light shall be furnished which will activate whenever the pressure is 70 psi or less.

EXCEPTION: Machines designed to handle railway cars must utilize train type brakes.

Comply: Yes ______ No ______ Remarks: ____

4. WHEELS, AXLES AND BEARINGS

4.1 Wheels and axles shall be in alignment and gage. No excessive vibration, wobble or eccentric action shall occur at any speed for which the machine is intended. Suspension shall provide for damage-free operation under maximum foreseeable operating stresses. Derail guards shall be incorporated near each wheel so that in the event of derailment the machine will not leave the rail.

4.2 Axles shall utilize double row, tapered roller type axle bearings. Where pillow block bearings are used, they shall be self-aligning, double row, non-expansion, roller type axle bearings, and shall have cast steel or ductile iron housings and their location is to be fixed to avoid movement.

Bearings selected for use shall have a radial load capacity of 20,000 hours B-10 life at 50 RPM and to exceed static-wheel load of the machine.

4.3 Equipment suspension must be sufficiently flexible so that any wheel can drop below the plane established by the other three wheels irrespective of any movement of the other three wheels. The amount of drop in inches shall be equal to, or more than the wheelbase of the machine in feet divided by eight.

Comply: Yes ______ No ______ Remarks: ____

5. INSULATION

5.1 Equipment specified as "insulated" shall be so constructed that no track circuit shunt can occur during work or travel.

5.2 Equipment specified as "non-insulated" shall not be converted from insulated design by bonding around insulating parts.

Comply: Yes ______ No ______ Remarks: ____

6. HYDRAULIC SYSTEM

6.1 Systems shall conform to the recommendation of the National Fluid Power Association (NFPA), American National Standards Institute (ANSI), and International Standards Organization (ISO).

6.2 Upon completion of manufacture and before any operation, all parts of the system shall be clean and free from contaminants. Threads, holes, cuts, flares, and machining must be deburred and cleaned.

6.3 Manufacturers must pre-filter all oil through a 10 micron absolute or finer filtering system on initial filling of hydraulic system.

6.4 All reservoirs shall be designed and constructed to prevent entry of foreign matter, including water.

a. Reservoirs of 10 gallons or larger shall include:

1. Baffles to separate intake and return lines to facilitate the separation of air and foreign matter from the hydraulic fluid, separate pump inlet from the settling portion of the tank and shall direct flow toward tank walls for maximum heat dissipation.
2. Access panels large enough for complete cleaning, inspection, maintenance and servicing of sump filters with an accessible means to employ the reservoir in the event the fluid is to be retained.
3. An air inlet breather which is of sufficient capacity to maintain approximate atmospheric pressure at maximum demands on the hydraulic system and to assure vacuum at pump inlet(s) shall not exceed 60 percent of pump manufacturer's recommendations. Air breather system shall be equipped with a 10 micron (B10 = 10) or finer filter, either cartridge or spin-on type.

   EXCEPTION: Sealed and pressurized system.
4. A filler with at least a 100 mesh screen, protected from external damage. Filler shall have a minimum capacity of 5 gallons per minute with 5,000 ssu viscosity fluid. Filler cap shall have a retainer that can be locked with a large padlock, similar to the type stated in 7.2.
5. A thermometer, in plain view, protected from damage, as near the intake line as possible, at the add point fluid level.
6. A static fluid level gauge to show full and add points protected from damage.
7. When immersion heaters are provided, it is preferable to utilize a type incorporating NPT threads so removal is possible without draining the reservoir.
8. Both the intake and return tubes shall be located below the minimum working fluid level so as not to cause cavitation or aeration.

6.5 Fluid temperature in the reservoirs shall not exceed 180 degrees F. at the reservoir outlet(s) while operating in a 110 degrees F. ambient. The minimum fluid temperature after fifteen (15) minutes warm-up or operation shall be at least 60 degrees with a 20°F ambient.

6.6 Pressure testing tee(s) shall be provided at locations to provide easy access for checking hydraulic pressures on all circuits. The tee(s) shall include a 1/4" NPT fitting with a male quick-disconnect fluid coupling.

6.7 Where failure of power plant or pump can immobilize components in a position which would prevent moving the machine, an emergency hand pump shall be optional in the circuit.

6.8 Fluid Filtration

a. Filtration shall not be less than recommended by manufacturers of the hydraulic system components.

b. In closed loop systems, filtration as recommended by the pump manufacturer will apply.

6.9 Fluid Conductors

a. Fluid conductors utilized in circuits operating under 3000 psi must use high pressure hose; SAE 100 R2 Type A, Hi-Impulse type with the following qualification requirements:

   1. Constructed with 2-wire braid reinforcement.
   2. Have a bursting pressure safety factor of 4:1.
   3. Tested to 300,000 impulse cycles at 250 degrees F.
   4. Have an operating temperature range of –50 degrees F. to +250 degrees F.
   5. Used with skive type reusable fittings.

b. All fluid conductors utilized in circuits operating over 3000 psi or in hydrostatic drive systems should use extra high pressure hose: SAE R12 type when utilizing hose sizes through one inch, which have the following qualification requirements.

   1. Constructed with 4-spiral plies of steel reinforcement.
   2. Have a bursting pressure safety factor of 4:1.
   3. Tested to 1,000,000 impulse cycles at 250 degrees F.
   4. Have an operating temperature range of –50 degrees F to +250 degrees F.
   5. Use with permanent, crimp type fitting for added reliability.

c. All fluid conductors utilized in circuits operating over 3000 psi or in hydrostatic drive systems should use ultra-high pressure, heavy duty, hi-impulse hose: SAE 100 R13 type when utilizing sizes above 1-inch, which have the following qualification requirements.

   1. Constructed with 6-spiral plies of steel reinforcement.
   2. Have a bursting pressure safety factor of 4:1.
   3. Tested to 500,000 impulse cycles at 250 degrees F.
   4. Have an operating temperature range of –40 degrees F. to +250 degrees F.
   5. Use with permanent, crimp type fittings for added reliability.

d. Pump supply hoses must meet the requirement of SAE 100R4, with reusable fittings.

e. Hoses shall not be:

   1. Flexed to less than their rated minimum bend radius.
   2. Installed or routed to expose them to temperatures above or below their rated operating temperature ranges.
   3. Subjected to any twisting, pulling kinking, crushing or abrasion.

f. Hoses shall be installed, routed and isolated where possible for proper support through clamping and/or brackets to avoid all of the above conditions, especially abrasion. If necessary, hoses subjected to excessive abrasion should be wrapped with an abrasion resistant wrapping or sleeve.

g. Hose is preferred. Where tubing is used, SAE 37 degree flared ends are required.

h. Tubing and piping shall be mounted to minimize vibration. Tubing shall have only gentle bends to change direction or compensate for thermal expansion. Bend radii shall not be less than three times ID.

i. Pipe threads are not recommended. Where they are used, they must be NPTF dry seal type.

j. Whenever practicable, valves shall be manifold mounted.

k. Galvanized or brass tube, pipe or fittings shall not be used.

l. Fittings shall be machined type.

m. Complete circuit diagram(s) showing the exact circuit(s) in use on the machine and large enough to be easily followed for troubleshooting must be furnished. Additional pictorial or cutaway diagrams may be shown.

n. Vacuum at pump inlet(s) shall not exceed 60 percent of pump manufacturer's recommendation or four inches of mercury, whichever is less, under standard conditions. Test opening shall be provided, utilizing 1⁄4" NPT port, sealed with a pipe plug.

Comply: Yes ______ No ______ Remarks: ____

7. ELECTRICAL SYSTEM

7.1 Electrical system shall conform to recommendations of the American National Standards Institute (ANSI) and the International Standards Organization (ISO), where applicable.

7.2 Upon completion of manufacture and before any operation shall begin, all parts shall be clean and free from scale, rust, water or any contaminants. All material and workmanship must be of good quality for the intended use.

7.3 Sequence of operation and electrical, physical, and schematic drawings showing the exact circuit(s) in use on the machine and large enough to be easily followed during trouble-shooting shall be furnished. Subsequent changes shall be covered by new drawings furnished to the customer.

7.4 Whenever practical, components shall be interchangeable.

7.5 Cable shall be routed to prevent exposure to damage. Thin wall conduit shall not be used except in a protected area.

7.6 DC Systems, when grounded, must have negative ground.

7.7 Standard, industrial grade, readily available components shall be used.

7.8 Electrical apparatus cabinets:

a. Cabinets shall be of steel construction. Clearance between walls and bare, "live" parts shall not be less than 1½ inches, unless affected interior surfaces utilize insulative plastic or fiber sheeting, where a potential in excess of 50 volts exists. Cabinets must be weatherproof.

b. Panels must be readily removable and parts easily accessible.

c. Adequate lighting shall be provided in cabinets.

d. Only pipes as electrical conduit or pneumatic tubing as used exclusively for control circuitry shall enter cabinet.

e. Interior metal surface walls must be painted with electrical insulating paint or covered with other approved electrical insulating material.

f. Nominal voltages used must be plainly shown on outside of cabinet.

g. All parts and groups of parts shall be identified by functions and clear, simple, exact reference to service diagram and parts list. Integral units such as circuit boards, should be considered as one part, if intended to be replaced as a unit. Complete parts identification shall be shown when practical, in order to minimize errors and time consuming reference to drawings or lists.

h. Wires must be equipped with good quality terminals and identified with permanent numbered markers, color coded when practical. Terminal posts must be plainly marked. Once used, a number or color code must not be reused for a different circuit. All wires must be neatly dressed and clamped.

i. Housings containing heat producing elements must be properly ventilated.

7.9 Motor control apparatus overload relays:

a. All fractional horsepower motors shall have fuse protection.

b. All 1/4 to 1 horsepower motors shall have automatic reset thermal protection within the motor itself, rather than in the starter, and fuse protection in the line.

c. Motors in excess of one horsepower shall have starter relays with built in thermal protection.

d. Overload relays shall be in each line of a 3-phase starter in ungrounded systems.

Comply: Yes ______ No ______ Remarks: ____

8. PNEUMATIC SYSTEM

8.1 Quotation will give: make, type, and output rating of compressor, size of reservoir; type and purpose of pressure regulation; and normal operating pressure.

8.2 System must contain air pressure gauge in easy view of operator and/or low air pressure warning indicator.

8.3 Standard air brakes hose SAE-J1402 Table A1 will be the only hose used anywhere in the system. Use of push-on barb type fittings are banned except for control circuits where space is critical and working pressures and abrasion is not severe and braking is not involved.

Comply: Yes ______ No ______ Remarks: ____

9. CONTROLS

All switches, valves, levers, controls, and adjustments used to start, stop or operate the machine shall be clearly labeled with weather and wear resistant plates permanently affixed to the machine or component. Controls used in continuous operation of the machine shall be within easy reach of the operator and shall not interfere with his view of the work. Instruments and gauges not inside a closed, lockable cab shall be protected from vandalism.

9.1 Engine must not start with controls in travel position.

9.2 A red emergency shutdown control shall be quickly accessible to operator.

Comply: Yes ______ No ______ Remarks: ____

10. CLEARANCE AND SAFETY

10.1 Components which may foul track, signal, crossing or other structures on or along a railway must have a positive mechanical lock in safe position for travel.

10.2 No component shall be less than three inches above top of rail.

10.3 All components which can be a hazard to operator, assistant or bystanders shall be protected with a shield or safety device.

10.4 Machines shall be equipped with a travel warning alarm which is energized by operation of the travel control for movement in reverse of the normal working direction. This shall be accomplished by use of a two-way selector switch (manually operated) which will establish the direction of travel that the alarm will be activated.

10.5 Handrails or grab irons will always be provided wherever it is intended that personnel mount equipment. Lowest step used for mounting the machine shall not be more than 12 inches above bottom of wheel elevation. Any area more than three feet above bottom of wheel elevation, where persons are expected to walk or pass, shall be protected by rigidly fastened handrails 42 inches high, with secondary rail at 24 inch height, and non-skid walking surface. A three inch kickboard shall also be provided to avoid accidental entry into potentially hazardous areas.

Comply: Yes ______ No ______ Remarks: ____

11. INITIAL PREPARATION AND SERVICING

Machine shall be delivered completely lubricated and serviced with all equipment needed for immediate operation, except for fuel. Water cooled engines are to be protected from freeze-up by a 50/50 solution of ethylene glycol base anti-freeze (rust inhibitive type) unless otherwise specified, and radiator marked or tagged to show make of coolant used and actual protection afforded.

Comply: Yes ______ No ______ Remarks: ____

12. PAINT

Manufacturer shall use paints which do not require periodic medical examinations or special equipment, other than passive respirators. It is assumed that no-lead paint with a lower percentage of solvent may be required and that exact colors formerly furnished may not be available. Manufacturer shall provide the colors listed under the "Standard Specification Number" which are consistent with those identified in the "Federal Standard 595-B, Colors Used in Government Procurement."

Manufacturer shall also utilize proper surface preparation, including primer, to provide a high quality, durable finish coat. General exposed parts of the Rail Guide Wheel Equipment shall be painted AREA Black, Spec. No. 17038, unless otherwise specified by a Railroad, EXCEPT AS FOLLOWS:

Equipment & Component Area	Color	Standard Spec. No.
Gasoline Tank*	Red	11086
Diesel Fuel Tank*	Green	14110
Hydraulic Reservoir*	Blue	15180
Coolant Tank*	Gray	16251
Wheels and Handrails	Black	17038
Lifting Lugs	Black	17038
Jacking Points or Pads	Black	17038
Safety Locks	Red	11086
Engine & Other Misc. Parts	Manufacturer's Option	

*NOTE: Where side or top of tank containing filler opening has an area greater than 144 square inches, manufacturer may elect to paint only 144 or more square inches at the filler opening with the required tank color. Name of fluid and words "CHECK DAILY" shall be stenciled on each tank in 1-inch letters. Total machine weight with all tanks filled shall be plainly marked on both sides of the machine in 1½ inch letters:

Weight ______ lbs.

Comply: Yes ______ No ______ Remarks: ____________________

13. SHIPPING

Equipment shall be constructed, prepared, and loaded so that it will withstand without damage, handling likely to be encountered during delivery. Valuable and easily pilfered parts such as batteries, tools, and loose small items shall be shipped in such manner as to resist pilferage.

Comply: Yes ______ No ______ Remarks: ____________________

14. PARTS AND INSTRUCTION BOOKS

Complete parts and instructions books shall accompany the machine. Additional sets of books shall be forwarded as follows: ____________________

____

Books shall contain complete and easily read diagrams of all systems on the machine and shall employ American Standard symbols and notations. Listings of commonly available parts shall include general descriptions as well as part numbers.

Parts book must also contain the comprehensive lubrication chart for the appropriate type machines.

Comply: Yes ______ No ______ Remarks: ____________________

15. REPAIR PARTS AND SERVICE

Continuous operation of this equipment is of utmost importance. Successful bidder must be able and willing to furnish service and repair parts promptly.

Comply: Yes ______ No ______ Remarks: ____

16. ADDITIONAL OPTIONS (To be completed by vendor or manufacturer.)

Insulated: Yes____ No____
Self Propelled: Yes____ No____
Shutdown: Yes____ No____
Turntable: Yes____ No____
Turntable Warning Light: Yes____ No____

NOTE: Options and accessories not covered on these specifications and which are available at extra cost shall be quoted separately in the bid proposal.

Comply: Yes ______ No ______ Remarks: ____

17. DELIVERY

Bid shall specify delivery date of all equipment offered. At the time of order, date will be reaffirmed or a new date established. A manufacturer's representative shall place equipment in service and instruct purchaser's operators, mechanics, and supervisors at a location to be specified by purchaser (not necessarily the machine delivery point).

18. MOUNTING OF RAIL GUIDE WHEEL EQUIPMENT TO VEHICLE

18.1 Mounting brackets shall be of high quality and designed to support the Rail Guide Wheel Equipment. In applications requiring the Rail Guide Wheel Equipment to support the vehicle, the mounting brackets must be of adequate strength.

18.2 Mounting brackets must bolt to frame of vehicle. Welding is not acceptable (per vehicle manufacturer standards).

18.3 Front tire of vehicle must have a 1½ inch minimum clearance above rail when front of vehicle is supported by the Rail Guide Wheel Equipment.

18.4 The vehicle frame must be of adequate strength for the intended application. Frame reinforcements may be required where the vehicle frame does not have the adequate strength. For these special applications, the Rail Guide Wheel Equipment Supplier must provide reinforcement to meet or exceed the original vehicle specification.

19. SETUP AND ALIGNMENT

19.1 During setup the vehicle shall be at curb weight with permanent attachments (spare tires, tool boxes less tools, utility box cranes, etc.).

19.2 Maximum tire inflation must be maintained at the lowest maximum pressure rating of the vehicle tire or wheel.

19.3 The frame of the vehicle must be square.

a. Diagonal measurement of the frame must be equal within 1/8 inch.

b. Wheelbase on both sides of vehicle must be equal within 1/16 inch.

c. Axles will be square with the frame within 1/64 inch per foot.

19.4 Vehicle wheels which require front tire contact with the rail must be checked for caster, camber, and toe in as recommended by the vehicle manufacturer after the Rail Guide Wheel Equipment has been installed.

19.5 Setup of Rail Guide Wheel Equipment shall be in accordance to the manufacture of the equipment.

a. Track gauge on the Rail Guide Wheel Equipment shall be set within 1/8 inch.

b. Guide wheel loads shall be set within the tolerance limits of manufacturer's recommended weight setting.

19.6 Vehicle and Rail Guide Wheel Equipment shall be aligned in accordance to the Rail Guide Wheel Equipment manufacturer's specifications.

19.7 Vehicle shall be track tested on rail according to the Rail Guide Wheel Equipment manufacturer's specification as a final check to insure proper operation.

20. NON-COMPLIANCE

These specifications are not intended to eliminate any product from the bidding. Where equipment does not comply, bidders shall clearly describe each deviation. These specifications are in full effect unless amended in writing by the purchaser. Purchaser reserves the right to reject any bids and the right to accept bids deviating from the specifications.

DATE: ______________________________, 19_______.

Specifications reviewed and completed by:

SIGNATURE: ____

TITLE: ____

COMPANY: ____

TO COVER: MACHINE: ______________________________ MODEL: _________________

Part 3—Reports and Records

Pages 27-3-1 through 27-3-13. Delete all forms (eleven total).

Proposed 1996 Manual Revisions to Chapter 29—Waterproofing

Part 2—Membrane Waterproofing

Pages 29-2-1 & 29-2-2. Articles 2.1.3 and 2.2.2. Insert reference to cold liquid-applied elastomeric membrane.

2.1.3 Types

(a) 7. Multiple layers of cold liquid-applied elastomeric membrane with an approved primer.

2.2.2 Primer

(c) Cold liquid-applied elastomeric membrane primer. Primer shall be of the type compatible with the substrate and membrane type as recommended by the manufacturer.

Page 29-2-4. Article 2.3.10. Cold Liquid-Applied Elastomeric Membrane. Add the following new article:

2.3.10 Cold liquid-applied elastomeric membrane

(a) The membrane shall be a 100% reactive spray-applied material.

1. Performance Requirements, Properties

Property	Requirements	Test Method
Membrane Thickness	2.5 mm (100 mils), min.	
Percent Reactive	100%	
Water Vapor Transmission Registered method used Desiccant method	6.6 grains/mm^2/24 h	ASTM E 96-93
Minimum Elongation at Break	80%	ASTM D 638-91
Minimum Tensile Strength	6.4 MPa (930 psi)	ASTM D 638-91
Adhesion to Steel	2.0 MPa (290 psi) min.	ASTM D 4541-89
Adhesion to Concrete	0.7 MPa (100 psi) min.	ASTM D 4541-89
Crack Bridging	Pass @ 25 cycles 1.6 mm (0.0625 inch) –26C (–15F)	ASTM C 836-89
Ballast Impact	No Damage	Test method as described in subparagraph 2.

2. Ballast Impact Test

   A. The waterproofing is placed on a deck on which a 600 mm (23.5 inch) diameter metal cylinder is placed. Ballast is put in the interior of the cylinder in direct contact with the membrane.

   B. A metal plate 300 x 300 x 50 mm (12 x 12 x 2 inch) which transmits a movement of 4 mm (160 mils) and a cyclic force of 41 to 125 kN (9.2 to 28.1 Kips) corresponding to a 25 tonne (27.5 ton) axle load is placed on the ballast.

   C. The test is carried out for 2 million cycles.

D. If no damage to the membrane is evident after completion of the test the sample has passed. If there is damage to the membrane after the test, the sample has failed, protective cover shall be installed before ballast is placed.

3. Certification

Manufacturer shall furnish certification that the material meets specification requirements.

2.9 Construction

2.9.1 General (add)

(c) Cold liquid-applied elastomeric membrane shall be applied when substrate temperatures are in the range of 0–40C (32–104F) providing that the substrate is above the dew point. The condition of the substrate shall meet the Manufacturer's recommendations and is approved by the Engineer. Material shall be sprayed on horizontal or vertical surfaces up to, around or into details.

2.9.2 Primer (add)

(c) Surfaces to be protected with a cold liquid-applied elastomeric membrane shall be given one coat of Manufacturer approved primer prior to the application of the membrane. The primer shall be applied by either spray, brush, roller or a method as approved by the Manufacturer.

2.9.3.4 Cold Liquid-Applied Elastomeric Membrane

(a) Surface Preparation

1. All concrete surfaces shall be cured for a minimum of seven days and shall be surface dry. Surfaces to be waterproofed shall be clean, smooth, dry and free of oil, grease and loose or foreign material.
2. The surface preparation shall be performed by means approved by the Engineer. The surface profile is not to exceed 6.3 mm (¼ inch) (peak to valley). Test method ASTM D 4541 shall be used to verify that the surface preparation meets the required adhesion/pull off values of 0.7 MPa (100 psi) for concrete and 2.0 MPa (290 psi) for steel surfaces.
3. Steel substrates shall be cleaned to a near white SSPC SP-10 specification or to a condition that exceeds the Manufacturer's minimum requirements. Special attention shall be given to welds, bolts, rivets, etc. so that preparation complies with Manufacturer's recommendations. Primer is to be applied within 4 hours of preparation.
4. Other methods of surface preparation recommended by the manufacturer may be used as approved by the Engineer.

(b) Application Procedures

1. Immediately prior to the application of any component of the system, the surface shall be dry. Any remaining dust or loose particles shall be removed using a vacuum or clean, dry, oil-free compressed air.
2. Where the area to be waterproofed is vertical, the system shall be capable of being sprayed at the specified thickness.
3. The membrane shall be carefully sprayed around and into drainage fittings to ensure proper runoff of water. Special care shall be taken with the spraying of the system to get full coverage along the sides and ends of girders, stiffeners, gussets, and over welds, bolts or rivets, etc.
4. Where the membrane is to be joined to existing cured material the new application shall overlap the existing material by at least 100 mm (4 in.). No preparation shall be neces-

sary unless the existing materials are dirty or contaminated in which case the overlap area shall be wiped with solvent (e.g., acetone).

5. The membrane shall be applied in a methodical manner to ensure proper coverage. Wet film thickness shall be checked once every 9 m^2 (100 SF).
6. If required by site conditions, or for application to small areas, or touch-up the membrane can be applied by brush or trowel in accordance with manufacturer's recommendations.
7. The membrane shall be fully cured before it is covered. Membrane shall be inspected prior to covering and any surface defects or damage shall be repaired in accordance with manufacturer's recommendations.
8. Protective cover in accordance with Subarticle 2.9.4.1, (c), 2. of Chapter 29 shall be installed prior to placing ballast if the cold liquid-applied elastomeric membrane does not pass the Ballast Impact Test or if recommended by the manufacturer.
9. Other application procedures may be used as recommended by the manufacturer and approved by the Engineer.

Proposed 1996 Manual Revisions to Chapter 32—Systems Engineering

Revise Chapter 32 as follows, including the change of title to "Engineering Management Systems" and the inclusion of a new Part 11, Computer Aided Drafting (CAD), formerly Part 2 of Chapter 11.

Manual for Railway Engineering Chapter 32—Engineering Management Systems*

Current From August 1, 1996 to July 31, 1997

Specification for Railway Engineering Management Systems

FOREWORD

Railroading has undergone significant change in all methods of operation in the last decade. Much of this change has caused all companies, and individuals within them, to reexamine their business objectives on a long term basis while also considering short term benefits. Particular emphasis has been placed on how to achieve these objectives in the most economical way possible. Automation systems and the thrust for a paperless office environment have played a key role in this change.

Automation systems; computer hardware and software, and communication technology; also undergo phenomenal change each year. The power of computers continues to increase significantly while the price and size are dramatically reduced making it now possible to economically place powerful computers or data terminals on virtually every desk or even in the pocket of field personnel. These changes have altered the tasks of Committee 32 in much the same way as railroad companies and agencies as a whole have been affected. The Committee has constantly studied and analyzed evolving concepts, technology and railroad practices in an attempt to provide and communicate to the AREA membership current and accurate information that can be used as a meaningful railroad engineering management tool.

It is with this concept of aiding AREA members in mind that Committee 32 has concluded that it is time to move beyond previous efforts, and begin the ardent preparation of permanent manual recommendations. The Committee's studies have shown that all major railroads and rail transit agencies, including many shortline railroads, have developed, or are now developing, automated engineering systems that encompass some or most of the railroad engineering management functions described herein. The majority of these systems have been developed as an independent endeavor. Each railroad has taken a slightly different approach and expended their major efforts on different engineering management functions. All railroads perform most functions listed, but using uniquely differing levels of either manual or automated techniques to process and store the information.

It is the intent of the Committee in preparing Chapter 32 to provide recommendations for the design and development of Railroad Engineering Management Systems, weather manual or automated, that will:

*The material in this and other chapters in the AREA *Manual for Railway Engineering* is published as recommended practice to railroads and others concerned with the engineering, design, construction and maintenance of railroad fixed properties, and allied services and facilities. For the purpose of this Manual, RECOMMENDED PRACTICE is defined as a material, device, design, plan, specification, principle or practice recommended to the railways for use as required, either exactly as presented or with such modifications as may be necessary or desirable to meet the needs of individual railways, but in either event, with a view to promoting efficiency and economy in the location, construction, operation or maintenance of railways. It is not intended to imply that other practices may not be equally acceptable.

1. Reduce duplication of effort
2. Demonstrate the need for single entry of data as close to the source of information as possible
3. Provide a unified, standard format and standard direction for systems development
4. Enhance compatibility and the interchange of information between Engineering and field locations; other railroad departments; and outside railroads, agencies, vendors or customers.

The purpose of Chapter 32 is to provide a guideline for recommended practice within the scope of study of Systems Engineering. Furthermore, it is hoped that this Manual Chapter will become a useful aid to a wide range of Engineering personnel, as well as members of the AREA.

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Part 11—Computer Aided Drafting—(CAD)[1]

1996

FOREWORD

The purpose of this part is to formulate specific and detailed rules for the development and utilization of Computer Aided Drafting (CAD) systems for the automation of the Engineering Management function, including defining recommended mapping, drafting and electronic drawing interchange standards.

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11.1 INTRODUCTION

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11.2 ENGINEERING DESIGN DRAWINGS

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11.3 MAPPING

11.3.1 Overview

11.3.1.1 Regulation

(a) Effective January 1, 1992, the Interstate Commerce Commission (ICC) eliminated Part 1263, Map Specification, from the Code of Federal Regulations and transferred significantly reduced map specifications to the property account instructions in the Uniform System of Accounts for Railroads, Part 1201. Due to improved technology in map making, the Commission ruled that it is no longer necessary to require railroad companies to maintain the detailed records previously required in Part 1263. However, because the Commission has a need for Class I railroad property records in rate, abandonment, merger and purchase proceedings and for accounting, audit and valuation purposes, these carriers are still required to maintain certain basic map information as herein described. This rule substantially reduced the regulatory burden associated with maintaining and filing property maps with the Commission. Additionally, this rule relieved Class II and Class III railroads of all map requirements.

(b) Class I railroads are subject to a five-part map specification incorporated as Instruction 2-21, Map Specifications, in Part 1201. The revised map specifications require Class I railroads to:

1. Maintain a current map of its rail property.
2. Furnish copies of such maps to the Commission upon request.
3. Maintain sufficient detail to show right-of-way, track and other important facilities.
4. Provide appropriate indices and titles.
5. Comply with generally accepted map principles.

11.3.1.2 United States National Map Accuracy Standards

(a) With a view of the utmost economy and expedition in producing maps which fulfill not only the broad needs for standard or principal maps, but also the reasonable particular needs of individual railroads, Standards of accuracy for published maps are defined as follows:

1. Horizontal accuracy: For maps on publication scales larger than 1:20,000, not more than ten percent of points tested shall be in error by more than 1/30 inch, measured on the publication scale; for maps on publication scales of 1:20,000 or smaller, 1/50 inch. These limits of accuracy shall apply in all cases to positions of well defined points only. Well defined points are those that are easily visible or recoverable on the ground, such as the following: monuments or markers, such as bench marks, property boundary monuments; intersections of roads, railroads, etc.; corners of large buildings or structures (or center points of small buildings); etc. In general what is well defined will also be determined by what is plottable on the scale of the map within 1/100 inch. Thus while the intersection of two road or property lines meeting at right angles would come within a sensible interpretation, identification of the intersection of such lines meeting at an acute angle would obviously not be practicable within 1/100 inch. Similarly, features not identifiable upon the ground within close limits, such as timber lines, soil boundaries, etc., are not to be considered as test points within the limits quoted, even though their positions may be scaled closely upon the map.

2. Vertical accuracy, as applied to contour maps on all publication scales, shall be such that not more than 10 percent of the elevations tested shall be in error more than one-half the contour interval. In checking elevations taken from the map, the apparent vertical error may be decreased by assuming a horizontal displacement within the permissible horizontal error for a map of that scale.
3. The accuracy of any map may be tested by comparing the positions of points whose locations or elevations are shown with corresponding positions as determined by surveys of a high accuracy. Tests should be made by the producing railroad, which shall also determine which of its maps are to be tested, and the extent of such testing.
4. Published maps meeting these accuracy requirements shall note this fact on their legends, as follows: "This map complies with National Map Accuracy Standards."
5. Published maps whose errors exceed those as specified in Article I, Section B, Paragraph 1-3 shall omit from their legends all mention of standard accuracy.
6. When a published map is a considerable enlargement of a map drawing (manuscript) or of a published map, that fact shall be stated in the legend. For example, "This map is an enlargement of a 1:20,000 scale map drawing," or "This map is an enlargement of a 1:24,000 scale published map."
7. To facilitate ready interchange and use of basic information for map construction among all Federal map making agencies, manuscript maps and published maps, wherever economically feasible and consistent with the uses to which the map is to be put, shall conform to latitude and longitude boundaries, being 15 minutes of latitude and longitude, or 7.5 minutes, or 3¾ minutes in size.

11.3.1.3 Objectives

(a) The specifications herein described propose the development and adoption of general guidelines for map creation and production by railway carriers. In keeping with the intent and spirit of the ICC regulation, the objective of this specification is to eliminate antiquated and restrictive cartographic standards for affected railway carriers. This specification should in no way be viewed as the definitive standard for railroad related cartographic practices. Those practices must be adopted and utilized by individual railway carriers to suite their parochial business needs and to fulfill existing ICC regulations. This specification permits flexibility for map development and production.

11.3.1.4 Scope

(a) This specification should serve as a flexible guideline to those railway carriers obligated under existing regulations to provide map and map related information to the ICC. Other railway carriers may wish to adopt the herein described standards to assure industry compatibility and for use as a resource management tool. In any event, the specifications should be considered broad enough to encompass the needs of individual railway business practices including historical and current uses as well as the application of new and innovative technologies.

11.3.1.5 Organization

(a) The suggested specifications are divided into the following major areas:

1. General Cartographic Practices—updated relative to ICC regulations and railway carrier operations.
2. Digital Mapping—as applied to rail carrier cartographic requirements.
3. Land Information—relative to mapping.

11.3.2 General Cartographic Principles

11.3.2.1 Intent

(a) In order that the requirements and interests of the railway carriers be best served, and that the needs of the ICC be provided for, it may be necessary to prepare certain maps of the property and have available methods of reproducing copies thereof to meet the requirements and demands as occasions arise. Although maps are not typically viewed as basic accounting records, they have proven to be an integral part of railroad property records. Maps are used extensively to identify and value rail property. Therefore, it is necessary to prescribe a uniform and consistent basis for identifying rail property. The specifications described herein substantially reduces the burden of maintaining and producing maps. The revised map specifications will provide railroad management greater latitude in developing and maintaining rail property maps.

11.3.2.2 Map Specifications

(a) Class I Railroad companies shall maintain current maps of its property and shall promptly record any changes that may take place.

(b) Class I companies shall furnish, on request, copies of maps showing its property as it exists on such date or dates as may be fixed by the ICC.

(c) Class I companies shall maintain planimetric maps that show right-of-way, track and other important facilities at a scale to show sufficient detail.

(d) Maps shall be indexed and titled to clearly indicate the specific area depicted.

(e) All maps shall be prepared in accordance with generally accepted mapping practices.

11.3.2.3 Classes and Titles

(a) Two general classes of maps may be made by the railway carriers:

1. Right-of-way and track maps.
2. Station maps.

11.3.2.4 Description and Purpose

(a) The right-of-way and track maps should be a true horizontal projection of the right-of-way tracks and other structures. The maps shall be made of materials of standard and durable quality, using conventional symbols and plain lettering.

(b) Station maps should be made when necessary to supplement the right-of-way and track maps at terminals or other locations where the properties of the carriers are so extensive and complicated that full and complete information cannot be shown on the regulation right-of-way and track maps.

11.3.2.5 Size of Sheets

(a) The maps should be made on sheets 24 inches by 56 inches. A plain single line border should be drawn on each sheet, dimensions inside of which shall be 23 inches by 55 inches. When more than one sheet is required to show the property, the maps should be made upon "matched marked" sheets in such manner as to require the minimum number of copies. The 24 inch by 56 inch map size is normally maintained by the railway carrier based on historical cartographic practice. However, the railway carrier is not restricted to the previous standard size and may elect to adopt any engineering size format to suit individual business requirements.

11.3.2.6 Scales

(a) The right-of-way and track maps should be made on a scale of 1 inch equals 100 ft., 200 ft., or 400 ft., as the importance of the maps may warrant.

(b) The station maps should be made on a scale of 1 inch equals 100 ft. or in complicated situations, 1 inch equals 50 ft.

(c) The railway carrier may elect to utilize a different scale at its discretion. However, such scale must be large enough to accommodate all features as may be required by ICC regulation.

11.3.2.7 Arrangements of Data

(a) At the railway carrier's discretion, the maps should be made with the zero or lowest numbered station at the left side of each sheet and should be plotted continuously from left to right. Where the use of the method would involve the abandonment of established survey station numbers of a railway, the plotting should be done in such a way as to preserve them, provided the maps for any given main line or branch are continuous in same direction between termini of main line or branch. The general direction of the center line of track should be as nearly as possible parallel to and half way between the long sides of sheets, so that the maximum space each side of plotted right-of-way lines may be available for showing adjacent topography and property lines and for delineation of such other features as may be deemed necessary. The maximum length of the main line roadway, represented on any one sheet of right-of-way and track maps between "match marks," should generally be 1 mile, if scale is 1 inch equals 100 ft., 2 miles if scale is 1 inch equals 200 ft., and 4 miles if scale is 1 inch equals 400 ft. (subject to 11.3.2.6).

11.3.2.8 Cardinal Points

(a) On all maps an arrow showing the true north and south line, as nearly as can be ascertained from existing records, shall be placed. This arrow should be less than 3 inches in length and shall have the letter "N" marked as its north end. On each end of each sheet there should be shown a pointer directing to a terminal or important station.

11.3.2.9 Indexing

(a) All right-of-way and track maps sheets should be numbered serially, beginning with sheet 1. The sheets representing valuation sections should form separate series and the valuation sections should be numbered serially with the letter "V" preceding the number. Index numbers should be in the lower right-hand corner of the sheet and enclosed in a plain single line circle measuring 1 inch in diameter. Valuation numbers should be in the upper half of the circle and the sheet number below separated by a straight horizontal line.

(b) The station maps should be given the same serial number preceded by the letter "S" as the sheet of the right-of-way and track map which they supplement.

(c) In case a right-of-way and track map sheet is supplemented by more than one station map, a subscript letter should be used after the number as follows: S 32a, S 32b, etc., where land and track features are combined; S-L 32a, etc., where land only is shown; and S-T 32a, etc., where the track features only are shown.

(d) On the right-of-way and track map sheets reference to all station maps should be shown by outlining the limits of station maps and giving the number of the station map sheets.

(e) Indexing is within the purview of individual railroads and should be maintained in accordance with stated ICC regulations {See 11.3.2.2 (d)}.

11.3.2.10 Title

(a) The title should be placed as near the lower right-hand corner as practicable. The following are generally accepted types of information which may be given therein:

1. Class
    a. Right-of-way and track map
    b. Station map

2. Corporate name of railway
3. Name of operating company
4. Name of railway division or branch line
5. Beginning and ending survey station numbers on sheet
6. Scale or scales
7. Date as of which maps represent the facts shown thereon
8. Office from which issued

11.3.2.11 Certification

(a) A certificate as to the correctness of all maps shall be executed and shall accompany such maps when submitted to the ICC.

11.3.2.12 Right-of-Way and Track Maps

(a) On these maps the following data should be shown:

1. Boundary lines of all rights-of-way, regardless of how acquired. The term "right-of-way" as herein used includes all lands owned or used by the carrier for common carrier purposes. Show width of right-of-way in figures at each end of the sheets and at points where a change of width occurs with station and plus of such points. Where known, boundary lines and dimensions of each separate track acquired should be shown. A schedule of land acquisitions for the land embraced on each sheet should be shown giving custodians reference, the name of grantor and of grantee, kind and date of instrument of title. Each separate parcel acquired should be serially numbered on the sheet and the corresponding number shall appear in the schedule reference. Where space is available this schedule should appear on the sheet to which it applies. In terminal locations or complicated situations where space on the sheet is not available, a separate schedule sheet properly referenced should be prepared to contain the information.

   The schedule should include leases to the company, franchises, ordinances, grants and all other methods of acquisitions.

2. Boundary lines of detached lands. The term "detached lands" as herein used includes:

   a. Lands owned or used for purposes of a common carrier, but not adjoining or connecting with other lands of the carrier.

   b. Lands owned and not used for purposes of a common carrier, either adjoining or disconnected from other property owned by the carrier.

   Show: Boundary lines and dimensions where known, distance and bearing from some point on the boundary line to some established point or permanent land corner where practicable, and separately on the map where the lands are not used for railway carrier purposes.

3. Intersecting property lines of adjacent landowners. Where known, show: The property lines of adjacent landowners, the station and plus of important intersections of property lines with the center line of railway carrier or other railway base line, and the names of owners of the land adjacent to the right-of-way.

4. Intersecting divisional land lines. Where known, show: Section, township, county, state, city, town, village or other governmental lines, with names or designations; the width and names of streets and highways which intersect the right-of-way; and the approximate station and plus at all such points of crossing or intersections with the center line of railway carrier or other railway base line.

5. Division and subdivision of lands beyond the limit of the right-of-way. Where known, show: The section and quarter section lines for a reasonable distance on each side of the center line or base line of railway where the land has been subdivided into townships and sections; such data as to divisions, tracts, streets, alleys, blocks and lots, where the land has been divided in some other way than by sections; the distance from the railway base line to permanent land corners or monuments; and the base line from which the railway's lands were located (center line of first, second, third, or fourth main track or other base line).
6. Alignment and tracks. Show: The center line of each main and sidetrack when such tracks are outside the limits covered by the station maps and the center line of each main track inside station-map limits; the length, in figures, of all sidetracks from point of switch to point of switch, or point of switch to end of track; all other railways, crossed or connecting, and state if crossing is over or under grade, and give name of owner of such tracks; survey station number at even 1,000 scale-feet intervals, and station and plus at points of all main line switches at points of curves and tangents and at beginning and ending points on each sheet; and the degree and central angle of main line curves.
7. Improvements. Show: Important facilities in general outlines and give station and plus thereof.
8. Topographical features. Where practicable show: Water courses, highway crossings, etc., give names where known and when highway crossings are over or under grade, so state.

11.3.2.13 Station Maps

(a) The purpose of the large scale station maps is to permit the showing of improvements in more detail than is practicable on the right-of-way and track map. Where the station property to be mapped is extensive and complicated, it should be delineated on two separate maps and should show the following:

1. All data relating to ownership of lands
2. Tracks and structures and external land boundaries.

(b) Where practicable, without sacrificing the clearness of the map, the two may be combined into one map. Show all information set forth under items 11.3.2.1 through 11.3.2.12, when inside of station-map limits. Tracks should be represented on station maps either by center lines or by rail lines.

11.3.2.14 Profile Maps

(Under Development)

11.3.2.15 Track Charts (Condensed Profiles)

(Under Development)

11.3.2.16 Data Base Utilization

(Under Development)

11.3.3 Digital Mapping

11.3.3.1 Overview

(a) Digital maps and automated cartographic information is generally formatted as either vector or grid data. Vector data describes area information as polygons and linear features as line segments. Grid data partitions land into a mathematical framework with locations specified by row and column numbers. The usual method of data collection from maps or similar source documents is by manually following the map feature lines on a digitizer table. Another approach to automate cartographic data acquisition is through use of scanning devices with either single-element detector or linear array.

(b) Storage of enormous amounts of digital map data requires an organized system for access and retrieval. A powerful interactive system is the primary working tool for digital data storage and manipulation. The software and hardware should work together to:

1. Create digital map data bases
2. Edit digital map data bases
3. Merge and manipulate digital map data
4. Selectively retrieve map detail levels either in graphic or textual/alphanumeric form
5. Produce reports and data tapes.

(c) Cartographic data should be entered and stored within the system in multiple detail layers/levels. The hardware and software should be powerful and extensive enough to support multiple layer/level scenarios. Each level of map data is stored on its own layer in conjunction with other like elements. This allows the retrieval of any number of desired combinations of levels. Each data layer/level should be digitized or scanned from all available original source maps. Special attention should be given to parcel "slivers" or information gaps.

(d) Finally, appropriate indices for maps and attribute data files should be established for each data base. A "key" or index map or equivalent should be developed to serve as a cartographic directory to all map sheets. Where appropriate or as may be required, alphanumeric "cross-key" and sequential systems should be utilized for cartographic levels and corresponding attribute data as applicable.

11.3.3.2 Layer/Level Concept

(a) Development of cartographic data base structures call for each level of map information to be stored in its own layer (level) in conjunction with similar data elements. Use of a number of different levels or layers is essential in order to provide the flexibility needed to meet the different requirements for varied user purposes. Information can be separated digitally into a maximum number of data levels which will permit efficient updating and precision plotting into a single composited base map sheet using data levels as may be required. In order to fulfill various user needs and provide a flexible analytical tool and data, the following serves only as suggested layer level designations:

1. Coordinate Reference Network Systems: Geodetic control and/or local state plane coordinate systems
2. Topographic Detail: Contouring, foliage, water systems and wetlands
3. Planimetric Details: Transportation systems, roads, building "footprints", other man-made features
4. Cadastral Detail: Property boundary lines
5. Leased Properties: Where railroad is lessor
6. Tenant Properties: Where railroad is lessee
7. Occupancies: Licenses between railroad and others
8. Zoning/Land Use/Taxing and Assessment Data
9. Deed and Conveyance, Rights/Interests Detail
10. Railroad Valuation Map Detail

(b) Additions or deletions of layers/levels can be made to accommodate individual business requirements.

11.3.3.3 Coordinate Network

(a) Geodetic control and state plane coordinates systems should comprise a separate level within the data base and should be utilized as the primary means for expressing and determining locations in continuous space so that shifts in parcel and feature positioning may be accurately adjusted, manipulated or analyzed (land parcels will be referenced spatially to man-made features). The accurate mapping of topographic, planimetric and cadastral and other land features requires a system of survey and cartographic controls which consists of a framework of points whose horizontal and vertical positions have been established and to which map details are adjusted and against which such details can be verified.

11.3.3.4 Topographic Detail

(a) A separate layer/level depicting topography need only be included for those railway carriers requiring this detail for specific uses. In such cases, contour intervals should be selected in conjunction with map scale, terrain relief and elevation data needs. Horizontal accuracy standards for large scale maps specify that 90% of points tested should be plotted with 1/30th inch of true position. Vertical accuracy standards specify that 90% of points tested should be shown in elevation within one-half of contour intervals used on map.

11.3.3.5 Planimetric Detail

(a) A separate layer/level should be established to delineate select culture detail and man-made ground features. These features include, but are not limited to, building "footprints", bridges, track, fences, catenaries, transmission lines, highways, and other structures and improvement. Planimetric details should be tied to coordinated points which are referenced to a horizontal and/or geodetic control network. The planimetric detail thus becomes a high accurate layer/level for precision position determinations, allowing for the employment of grid oriented mapping techniques {See 11.3.2.12 (a) 6 and 11.3.2.12 (a) 7.}.

11.3.3.6 Cadastral Detail

(a) The cadastral detail layer or level depicts spatial positioning of property boundary lines in relation to other features shown on the planimetric layer/level and as related to the coordinate network level. This level should provide for a timely, complete and available inventory of all existing land parcels. Cadastral (or property) boundaries should be viewed as lines which connect points having unique identities and records, and through which these boundaries can be physically located on the ground. Those boundaries can be expressed by points or corners, and straight or curvilinear lines {See 11.3.2.12 (a) 1 through 11.3.2.12 (a) 5}. Each parcel of land depicted on the cadastral level should have a unique identifier for correlation to attribute records. These unique parcel identifiers should provide the means by which to "link" the parcel to attribute data containing information about land ownership, use, value, area and so forth. Parcel identifiers can be developed or expressed in terms of one or a combination of the following:

1. Abstract Identifier: tract index based on a sequential numbering system.
2. Name Related Identifier: identifier for individuals or legal entities having an interest in a parcel of land.
3. Alphanumeric Identifier: random letters and numbers identifier.
4. Location Identifiers:
   a. Hierarchical—based on graded series of political units such as federal, state, county, city, town, ward, precinct, etc.
   b. Coordinate—relates parcel to reference grids, either through the use of geodetically derived latitudes and longitudes, or through the use of arbitrary or state plane coordinate systems.
   c. Hybrid—any combination of location identifiers.

11.3.3.7 Lease Properties

(a) Lands leased to individuals or other entities should be delineated on a separate layer/level. Leased parcels should be correlated to the planimetric and cadastral levels for the purpose of ascribing accurate representations of affective properties. Metes and bounds (bearing and distance) descriptions devised for the leased areas should be registered to the coordinate network. Attribute data should be tied to cartographic representations through the use of unique parcel identifiers {See 11.3.2.12 (a) 1 through 11.3.2.12 (a) 5.}.

11.3.3.8 Tenant Properties

(a) Lands leased by the railroad from individuals or other entities should be delineated on a separate layer/level. Tenant properties should be correlated to the planimetric and cadastral levels in order to ascribe accurate representations of affected parcels. Property boundary line descriptions (metes and bounds) should be registered to the coordinate network. Attribute data should be linked through use of unique parcel identifiers to cartographic representations of tenant properties {See 11.3.2.12 (a) 1 through 11.3.2.12 (a) 5.}.

11.3.3.9 Occupancies

(a) Occupancies include pipe and wire, sidetrack, crossing and similar license agreements affecting railway carrier properties and rights-of-way. Precise positioning of occupancies in relation to railroad facilities is an important record keeping need. Consequently, these occupancies should be delineated on a separate layer/level correlated to cadastral and planimetric levels in order to develop accurate representations. All descriptions and locations should be registered to the coordinate network. Applicable attribute data should be linked through the use of unique parcel identifiers to cartographic representations of occupancies. Occupancies should be completely and accurately delineated and inventoried to satisfy individual railway requirements, as well as public and safety needs.

11.3.3.10 Zoning/Land Use/Taxation and Assessment Detail

(a) Assessment, land use, and zoning details can be developed as a separate cartographic level (with accompanying attribute details) or encompassed within the cadastral level's attribute file {See 11.3.2.2 (d) and 11.3.2.12 (a) 1 through 11.3.2.12 (a) 5.}. If shown as a separate cartographic level, assessment, land use, and zoning details should be shown as special or colored boundary lines to differentiate between varying classifications. Assessment parcels should be shown for railroads, whereas land use and zoning details should be delineated for both railway carriers and adjacent properties. Assessment, land use, and zoning cartographic details are retained at the discretion of the railway carrier based on individual business requirements.

11.3.3.11 Deed and Conveyance, Rights/Interests Detail

(a) A separate level/layer can be established for deed and conveyance, and rights and interests with pertinent attribute files. This level should show inconveyances (parcel/land acquisitions), outconveyances (land sales), property interests, and other rights (aerial, surface, subsurface and operating). This level should sequentially depict deed, easement and other legal descriptions. These descriptions should be correlated to cadastral and planimetric levels in order to ensure accurate representations and should be registered to the coordinate network. Applicable attribute data in terms of title histories, execution and recording information, agreements and similar data or documentation should be linked through the use of unique parcel identifiers to cartographic representations {See 11.3 2.12 (a) 1 through 11.3.2.12 (a) 5.}.

11.3.3.12 Railroad Valuation Detail

(a) Unless incorporated within the planimetric detail level (11.3.3.5) or on a text level (11.3.3.13) separate level for valuation map detail should be developed to include the centerline of mainline and side track and the length of side tracks. Length of side track measurements should be shown from point of switch to point of switch, or point of switch to end of track. Additionally, survey stations should be shown at even 1000 foot intervals as follows: for station and plus designations at points of all main switches; at all crossing and bridges; at all structures and buildings; at point of curvature and tangency; and at beginning and end points (match or seam lines) on each sheet.

11.3.3.13 Lettering and Fonts

(a) Unless shown on each individual detail level, lettering should be shown on a separate level. Attention must be given to alignment, spacing, size, style, form, and locating for all lettering appearing on all map detail levels.

11.3.3.14 Economics and Benefits

(Under Development)

11.3.3.15 Aerial and Ortho Photography

(Under Development)

11.3.4 Land Information

11.3.4.1 Overview

(a) Effective land management and digital mapping encompasses a broad range of activity revolving around land resource assessment, planning, and regulation processes. Detailed land data on an individual parcel basis is required for day-to-day operations and the administration of buildings and lands.

(b) Comprehensive data base development requires the gathering and processing of vast amounts of information from numerous internal and external sources. This information is used to locate and identify parcels, describe land and structures erected on it, and meet specific system user needs. Data collection and structure development should address the organization's broad based purposes including comprehensive real property or right-of-way inventorying, accurate parcel valuation, equitable real estate assessment, and maximum utilization of land. Each data base should be individually structured to accommodate railway carrier business requirements. Attribute data should be linked to cartographic elements through the use of unique parcel identifiers. This will provide a continuously updated comprehensive record of land at the parcel level.

11.3.4.2 Planimetric Details

(a) Attribute data files relative to planimetric details includes information concerning tracks, buildings, structures (bridges, viaducts, etc.), electrical, communications and signal transmission networks and other physical man-made features. Attribute data for planimetric details are defined in terms of size, shape, design characteristics, construction materials and quality, and age and condition as follows:

1. Size is identified in terms of total area, volume, height, leasable space and/or clear span.
2. Shape is described in terms of a ratio of area to perimeter and number of corners or by matching shape or perimeter with a generalized pattern (rectangular, L shaped, G shaped or H shaped).
3. Design characteristics describe intended or designed use, arrangement and type planimetric detail and period of construction.
4. Construction materials include those elements used in the construction of foundations, frames, floors, walls, roofs and other structural features.
5. Construction quality refers to the composite characteristics of construction. This encompasses the cumulative effects of workmanship, coastlines of materials, individuality of design, and specific costs of structures.

6. Age and condition, the effects of wear and tear either in chronological age or "effective age" (adjusted for condition and remodeling) and the remaining economic life can be a part of the attribute file for a specific planimetric detail.

(b) Additionally, the value of planimetric details can be encompassed within the attribute data files. Attribute data should either be "attached" to each planimetric feature depicted on the map or developed in conjunction with the creation of a planimetric symbol. Planimetric attributes can be included within the cadastral detail attribute data files.

11.3.4.3 Cadastral

(a) The cadastral attribute data file is composed of demographic information concerning the location, shape and dimensioning of real property holdings. This should include, but is not limited to, area (square footage/acreage), ownership names, premise address, map/parcel identifiers, applicable file numbers, grantor/grantee data (optional), mile posting/val stationing, valuation and assessment data, date of acquisition, ownership type, zoning, and land use.

(b) The cadastral attribute data files should provide a complete and available inventory of all existing land parcels encompassing a distinct division between operating and non-operating properties. Parcel sizes should be recorded including dimensions (lot frontage and depth), total land area versus useable land area, setbacks, shape, and topographic soil characteristics. Land uses and improvement data should also be included.

(c) Additionally, the cadastral attribute data file should encompass locational and neighborhood characteristics. Locational characteristics are external to land parcels and involve view, presence of nuisance, and distance to services (communications, utilities, water, etc.). Neighborhood characteristics are elements such as physical barriers, geo-political boundaries and cultural aspects.

(d) The cadastral attribute data file should be "attached" to a unique identifier (coordinate or other) as depicted on the corresponding railroad map.

11.3.4.4 Lease and Tenant Properties

(a) Lease and tenant property attribute data files should be handled in the same manner a cadastral attribute data. However, in addition to general information (locations, size, shape, value, etc.), detailed data concerning the area of lease, term of lease, date executed, leasee, amount of lease, payment schedule, incremental lease costs and other lease or tenant data variables should be included. Such data elements or files should be "attached" to the appropriate map through use of a unique identifier (account number, coordinate point, etc.).

11.3.4.5 Occupancies

(a) Occupancy attributes files (pipe, wire, sidetrack, crossing, or similar license agreements) should be developed like those for cadastral and lease and tenant level details. In addition to the data elements normally depicted in the cadastral and lease and tenant files, occupancy attribute files should include the type of occupancy (pipe, wire, sidetrack, crossing, etc.), the term of license and exact location.

(b) A description of the license should also be included within the attribute data file. This description should encompass area and linear measurements as follows: if a wire crossing—the length, number of poles, conduit type and type of transmission (communications, electrical, etc.); if pipe—type, size, length, pressurized/non-pressurized; if sidetrack and other types of area—data relative to specific nature or type of license. The occupancy attribute data file should be attached to the map through an identifier.

11.3.4.6 Zoning, Land Use, Taxation/Assessment Detail

(a) A separate attribute data file can be created for each affected property and should detail zoning, land use, taxation/assessment information. Zoning data should be retained to determine whether land can be developed and how property can be used. A record should be retained of planning actions, zoning changes, the impact of master plans on affected and adjacent properties, and urban renewal or redevelopment requirements. Land use data should include land use codes, business licensing history, evaluations of proposed development, and site selections of proposed developments. Taxation and assessment information is necessary to support financial assistance requests and will aid in the administration of equitable real property taxation and assessment.

(b) The data recorded with this attribute file should contain site and improvement characteristics, factors and methods used in appraising or valuating properties, cultural and environmental conditions and marketing data (sale prices and terms, rental revenues, operating expenses, building costs and valuation models). Zoning, land use and taxation/assessment information can be included as part of the cadastral attribute data file.

(c) If maintained as a separate data file, the zoning, land use and taxation/assessment attribute file should be linked to the appropriate map level detail through use of a unique identifier.

11.3.4.7 Deed and Conveyance Information

(a) A separate detailed attribute data file should be developed for each map. Information contained in this file should include title and transfer information, identification and nature of property interests (simple, fee simple, aerial, subsurface), type of transfer (deed, land contracts, condemnation, wills, etc.) and terms of sales and/or transfers. Also, information concerning recordation, execution, railroad recordation, and the purpose of the transfer should be included. This attribute data file should be linked to the appropriate map level through use of an applicable identifier.

11.3.4.8 Data Base Development

(a) Relationships between data elements should be identified for system design, implementation and maintenance and for the coordination of related user requirements with data element definitions. System analysis begins with interviews with user groups to determine functional responsibilities, informational needs, analytical/decision making processes and the availability and condition of existing data sources. A concept of system design (what the system should or will be) should be created to support a decision for either internal or external system development. In implementing the system, consideration must be given to making sure that it performs in the manner in which it was designed.

11.3.4.9 Symbology Specifications

(a) Symbology specifications include line construction specifications, symbol construction criteria and the identification of detailed instructions coded into the symbol file. These instructions result in an appropriate graphic image display on the graphic CRT (cathode ray tube) and in accurate plottings of the graphic element.

(b) Symbols representing items for current source documents should be:

1. Evaluated to provide information concerning the quantity and conditions surrounding the use of individual symbols.
2. Analyzed to determine whether elements can be consolidated into a common representation, eliminated if not of value, created if required and not currently existing, or displayed or depicted with a more appropriate representation.

(c) Uniform symbology permits the railway carrier to efficiently maintain each data base.

11.4 DIGITAL DOCUMENT MANAGEMENT

(Under Development)

11.5 DATA AVAILABILITY AND SOURCES

(Under Development)

Proposed 1996 Manual Revisions to Chapter 33—Electrical Energy Utilization

Part 2—Clearances

Page 33-2-11. Article 2.2.10. Correct the tolerances in this article to read as follows:

2.2.10 Tolerances in the Catenary System and Position of Track

Allowances should be made in addition to the passing (P_A) and static (C_A) clearances, for civil engineering and wire installation irregularities in determining the vertical structure opening.

Under minimum clearance conditions, the catenary system should be maintained to ±0.5 inches (13 mm) of the design level at the supports. It is recommended that an extra 0.5 inch (12 mm) allowance be added to the tolerance for contact wire height above load gauge at normal clearance conditions.

The position of the track is assumed to be maintained within the following tolerances of the design position at overhead structures with limited clearances, as shown by the T_3 dimension on Figure 1, and by the L dimension on Figure 2:

Main line height at minimum clearance	± 0.5 inches (13 mm)
Main line height at normal clearance	+ 1.0 inches (25 mm)/–0.5 inches (13 mm)
Main line lateral alignment	+ 1.0 inches (25 mm)
Tunnels, bridges and overpasses height	+ 0.5 inches (13 mm)/–0.0
Track lateral alignment at tunnels, bridges and overpasses	± 0.5 inches (13 mm)

Part 6—Power Supply Requirements Railroad Electric Traction Systems

Page 33-6-1. Replace current Part 6 with the following revised Part 6 and applicable Table of Contents.

Subcommittee 6—Power Supply and Distribution Part 6—Power Supply and Distribution Requirements for Railroad Electrification Systems

Part 6—Power Supply and Distribution Requirements for Railroad Electrification Systems

6.1 GENERAL

Part 6 of this Chapter describes the studies and designs recommended for implementation of modern ac electrification systems for high speed, commuter and freight railroads. Recommendations for dc transit power supply systems are presented in Chapter 12 of this Manual.

Any electrification project requires coordination among all engineering designers, including those for traction power supply and distribution systems, signaling and communication systems, locomotives, and multiple unit cars. The track work, civil/structural/architectural, and maintenance facilities designers and the railroad operations and maintenance departments should be involved in design as soon as possible after the project commencement. It is desirable that all design groups become aware of each other's design and financial constraints as early as possible to optimize their designs within the technical and economical context of the overall project.

It is recommended that the electrification system designers initiate early meetings with technical staff of the power utilities to establish coordination channels and to discuss the impact of electrification on their systems. This will allow the utilities to plan in advance for any provisions necessary to deliver power to the new electrification system and to prepare for gradual increase in project involvement.

6.1.1 Environmental Considerations

Each electrification project is likely to require an Environmental Impact Statement (EIS). The factors which will have to be addressed include protection of natural resources during construction and operation, soil erosion and sedimentation, air quality improvement, noise and vibration and aesthetic impact. Adverse effects on the environment should be limited as much as is economically possible. It is recommended that the EIS be prepared as soon as possible after project commencement in order to avoid regulatory delays.

6.1.2 Electromagnetic Fields

Each electrification project is likely to require some investigation into the impact of electromagnetic fields (EMF). At the present time, the United States has no national standards which establish acceptable limits of electromagnetic field strengths. Several states have adopted guidelines and regulations and these should be followed in states where electrification is planned. Also, it is recommended that the designer follows guidelines established by other professional or regulatory organizations, such as the World Health Organization (Environmental Health Criteria), the International Non-Ionizing Radiation Committee (INIRC) of the International Radiation Protection Association (IRPA), and the American Conference of Governmental Industrial Hygienists (ACGIH).

6.2 SYSTEM CONFIGURATION

Electrification systems must be designed to be compatible with the locomotive/MU car propulsion systems, railroad signaling and communication systems, and the power utility systems.

6.2.1 Major Components of a Typical System and System Types

A typical configuration of an ac electrification system is shown in Figure 6.1. The system substations are located along the route at predetermined spacings and receive power at high voltage from utility systems. The high (primary) voltage is transformed by traction power transformers in the substations to provide a nominal voltage of 12.5 kV, 25 kV or 50 kV along the catenary system. Refer to Part 3 of this Chapter.

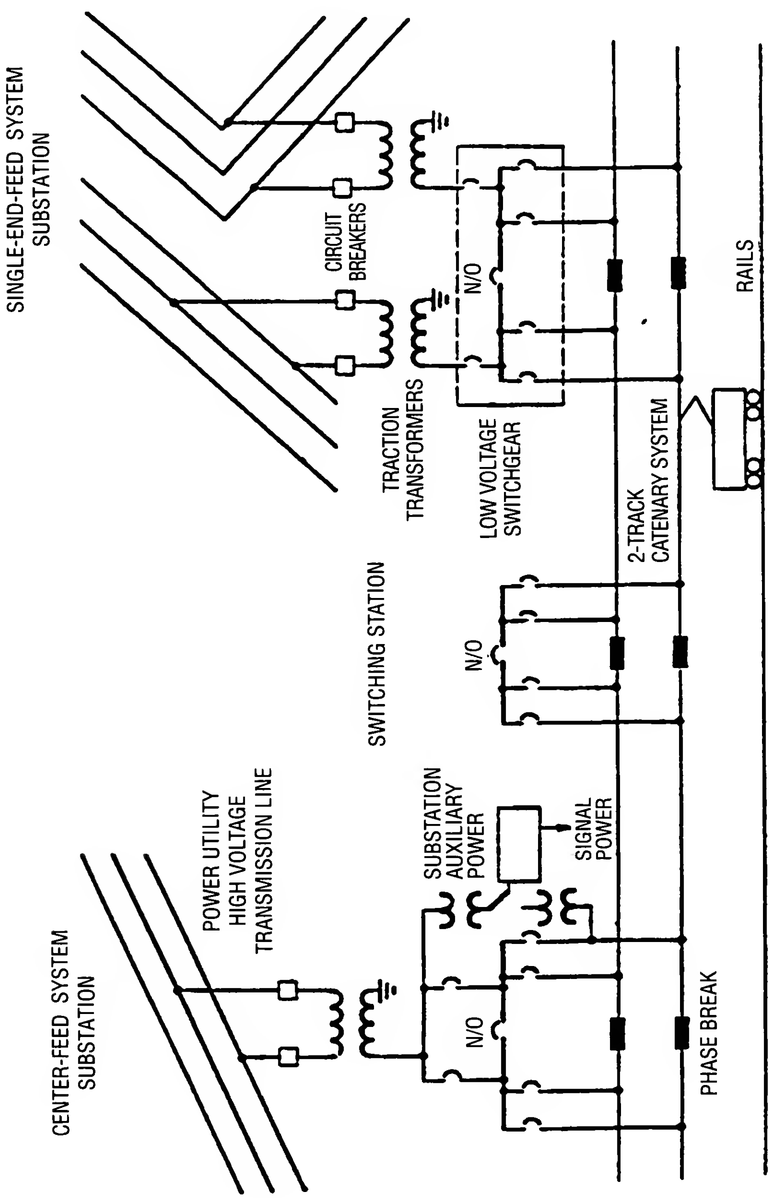


Figure 6.1. Components of a Typical Electrification System.

The traction power distribution system can be supplied with electrical power by substations with one or two traction power transformers. A system with one transformer per substation, called a center-feed system, is the simplest and most cost effective configuration and is recommended for initial consideration. A system with two transformers per substation, called a single-end-feed system, is recommended when substations are required to supply an unusually high power demand or in the event that their special location (near a tunnel, large yard or end of an electrified route) warrants a higher reliability of supply. An electrification system will usually be supplied by a combination of both types of substations.

System configurations using autotransformers are important alternatives to the above arrangements. Autotransformer systems use an along-track feeder to transmit energy to the trains and therefore can achieve a longer substation spacing at the same traction voltage as systems without autotransformers. The feeder currents, for the most part, oppose the catenary currents, and therefore the system can reduce electromagnetic interference. Consistent with the basic feeding system types described above, the autotransformer systems can be center-feed and single-end-feed as shown in Figure 6.2.

Traction power is supplied to the power distribution system via low voltage (secondary) switchgear. The switchgear circuit breakers are recommended to have either vacuum or sulfur hexafluoride (SF6) interrupters. The distribution system consists of a catenary system and, in the case of the autotransformer system, an along track feeder. Both systems are also equipped with ground (static) wires.

6.2.2 Normal and Emergency Operation

Figures 6.1 and 6.2 show the supply systems for normal operating conditions. During system emergencies, substation transformers must be capable of supplying their own section and an adjacent section. The geographic extent of feeding by the transformers in service depends on the type of supply configuration. In the event of a substation failure in the center-feed system, continuity of supply is maintained by the two neighboring substations. This is achieved at switching stations by closing the bus-tie circuit breaker. During a transformer failure in the single-end-feed system, the continuity of supply is achieved by closing the substation bus-tie breaker. When an entire substation in the single-end-feed system is out of service, the power supply is maintained from the adjacent substations by closing the switching station bus-tie circuit breakers.

With autotransformer systems the same basic switching procedures are followed as described above. However, because the feeder system is sectioned in the same way as the catenary system, feeder switching in the substations and switching stations is also required.

6.3 ELECTRIFICATION SYSTEM STUDIES

Electrification studies are essential for the traction power supply system design, power utility studies, interference studies, and for electrification system detailed design. The purpose of electrification studies is to develop basic system data such as substation power demands, catenary currents and catenary voltage profiles to enable the designer to correctly locate the traction power supply substations and to establish all major design parameters for the supply system.

6.3.1 Train Traffic Types

Railroad traffic can be broadly divided into two major types, freight and passenger. Freight trains operate at relatively low acceleration rates and their power demand during acceleration and cruising can be of the same order of magnitude. Passenger trains operate with higher acceleration rates and therefore their power demand is relatively high during acceleration and decreases considerably once the train has attained its normal cruising speed.

Further factors such as train weight, maximum speed, track curves and grades, density of traffic, railroad operating practices, and locomotive/MU propulsion equipment design affect the power demand requirements. It is recommended that discussions be held with the railroad operations department and locomotive/MU engineer-in-charge to identify means of limiting or reducing power demand to lower the cost of the traction power system.

6.3.2 Characteristics of Traction Power Demand

Because of frequent train acceleration, deceleration and changes in track profile, the train power demand has a highly fluctuating pattern. Although most modern locomotives have a power ramping circuit that prevents near instantaneous power changes, the resulting current can cause harmful pulsating forces in the substation equipment and voltage flicker in the utility system.

Propulsion systems on modern locomotives and multiple unit vehicles are thyristor controlled and, therefore, generate harmonic currents. The harmonic currents cause corresponding harmonic voltage drops which distort the traction and utility voltages. The harmonic distortion of system currents and voltages causes increases in electrical equipment heating and energy losses. In severe cases, the harmonics may cause system resonance and interference with communications and signaling circuits. Low locomotive power factor (refer to Figure 7, Part 8 of this Chapter), can cause excessive voltage drop along the catenary system. In the event that the power factor is not corrected in the substations, the utilities may charge the railroad a penalty as a part of their rate structure.

The adverse effects due to the traction power demand tend to be reduced with higher traffic density. Generally, with more than one train between two adjacent substations, the magnitude of current fluctuations at each substation are lower, the harmonics are of lower magnitude due to cancellation effects, and the average power factor is higher.

6.3.3 Load Flow and Power Demand Study

The Load Flow and Power Demand study is the basic study establishing all major design parameters needed for detail design. Generally, the locations of suitable utility power supply sources with proximity to adequate substation sites are identified first. Then, the system feeding configuration, electrification voltage and power distribution conductor sizes are provisionally selected.

In order to evaluate traction power system performance, it is recommended that designers calculate the voltages at each train, currents in each catenary conductor, and power demands at each substation. These calculations are required over sufficiently small increments of time, in the range of 5 seconds to 1 minute, so that the effects of fluctuating train power demand can be identified and assessed against the electrification system parameters.

Since a very high number of calculations is required, the use of a computer with an appropriate simulation program is recommended. Such programs use as input the results of Train Performance Calculations (refer to Part 2 of this Chapter), and electrical system data. The electrical data include electrification voltage, catenary and rail impedances, substation impedances and locations, and representation of the power utility system at the substation connection point.

Based on the computer run results, adjustments of the initial system parameters may be desirable to obtain a more satisfactory technical performance or more cost effective design. System design is an iterative process and several such computer simulations may be required to develop a well integrated, optimized and cost effective system.

6.3.4 Substation Spacings

It is often cost effective to obtain the longest possible feeding distance for each substation subject to the Power Utility and Interference Studies (refer to Sections 6.4 and 6.5 below). The factors influencing substation spacing can be broadly divided into two main categories - power supply fac-

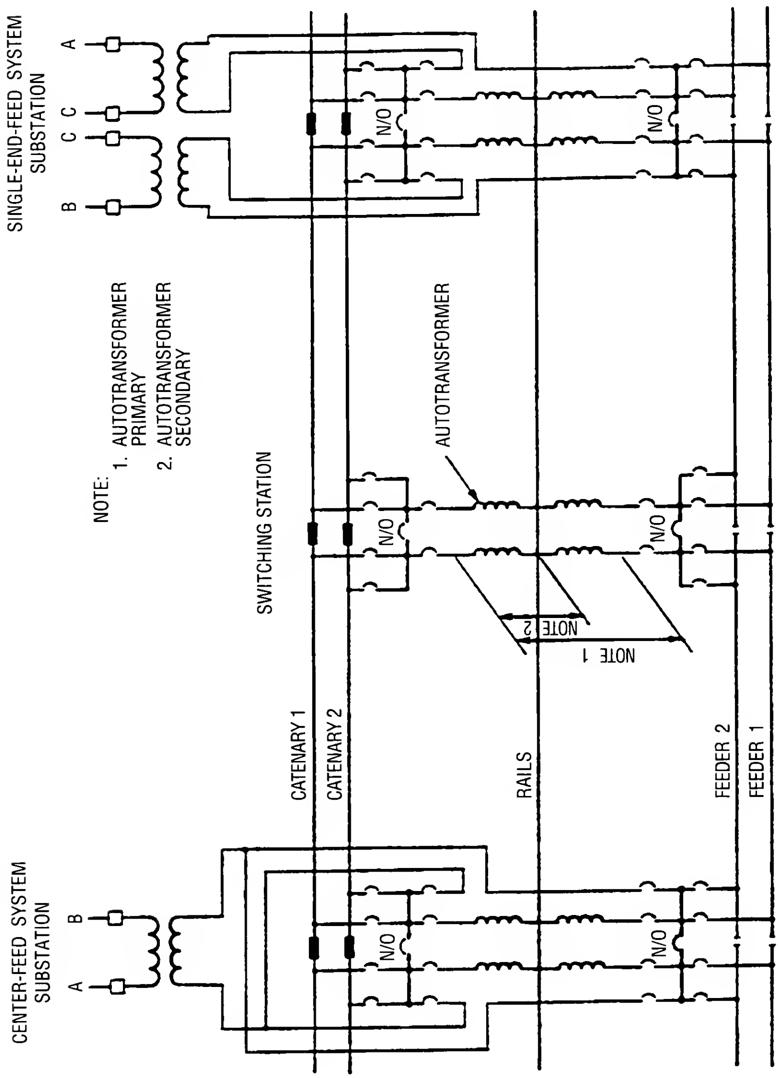


Figure 6.2. Components of an Autotransformer Electrification System.

tors and railroad factors. The power supply factors include electrification voltage, feeding configuration type, substation fault level magnitude, and catenary conductor sizes. The railroad related factors include train power demand, train headways, and locomotive/MU car power factor.

Substation spacings are based on voltage drop calculations. In the overall system comprising the utility network, substation transformers, feeders, catenary conductors, and rails, every train should have adequate voltage at the pantograph. This means that the train voltage should not decrease below the Normal Lower Voltage Limit values as recommended in Part 3 of Chapter 33 under the defined normal operations and should not decrease below the Emergency Minimum Operating Voltage during outage conditions.

The actual substation locations will often depend on the availability of suitable sites and adequate utility supplies and this, in turn, will influence the substation rating and the size of the distribution system conductors. By optimization of the power supply and railroad factors, it is often possible to use existing power utility facilities and limit the financial expenditures and environmental problems associated with building new transmission lines. Because the power supply system is a significant cost item on any electrification project, cost optimization is an important part of any system study effort. It is recommended that designers consider several technically acceptable designs during system studies, compare their estimated costs and carry out performance/cost trade-off evaluations to identify the most acceptable system alternative.

6.3.5 Substation Rating

Substation transformers are required to have sufficient short-time, medium-time and continuous power ratings in order for the power system to supply the fluctuating traction power demand. To determine the short- and medium-time power ratings, it is recommended that maximum RMS power demands be calculated over 5-minute, 30-minute and 2-hour time intervals at each substation. To establish the continuous rating, power demands over a longer time interval, 6 to 24 hours, may need to be calculated. The interval length depends on the train headways and mixture of train types.

6.3.6 Catenary Conductor Sizes

The size of catenary conductors proposed for an electrification project should be verified by calculating the overhead system ampacity as recommended in Part 4 of Chapter 33, using climatic data relevant to the specific project geographic location.

6.4 POWER UTILITY STUDIES

The source of electrical power for traction substations will be local or regional power utility companies. From the railroad point of view, the traction substation supply should be sufficiently reliable so as to avoid frequent emergency operation and should have sufficient capacity to facilitate a good voltage profile along the catenary system. In considering utility requirements, the utility system must have sufficient spare capacity to supply the new load, and the effects of the traction load must not disrupt operation of the utility system and affect the equipment of other utility customers.

When selecting substation supply points it is recommended that comprehensive studies of the utility system be performed to confirm that the traction loads can be supplied with no adverse effects on the utility and traction power systems.

6.4.1 Utility Voltage Level

Preferable voltage levels for substation input are in the range of 115 kV to 230 kV. Systems below 115 kV are not usually the first choice as the line capacity and fault level may be too low. Using supply at voltages above 230 kV is generally not cost effective.

6.4.2 Utility System Types

In general, the transmission system can be of the radial, loop or network type. A radial system consists of a single transmission line supplying a substation. A loop system is usually formed by the interconnection of radial feeders. The network system is generally a result of expansion of radial and loop systems. The network system is recommended over the loop system for substation connections and the loop system is recommended over the radial system. Substations with two transformers should be supplied wherever feasible by two independent lines originating from different parts of the utility system.

6.4.3 Voltage and Current Unbalance

A single-phase traction load unbalances the three-phase utility system and causes negative sequence current flows in power generators and three-phase motors. These currents produce additional heating in the machine rotors.

ANSI C50.13 and NEMA MG-1 standards cover continuous and short time negative sequence current capability for cylindrical-rotor synchronous generators. Rotating machines should withstand the effects of continuous negative sequence current of 5-10 % of rated stator current without injury. On a short time basis, the magnitude of product I22 t should not exceed 40 for motors, hydraulic turbine or engine driven generators, 30 for indirectly cooled turbine generators, 10 for directly cooled generators up to 800 MVA and 5 for some very large machines, e.g. 1,600 MVA. I2 is the negative sequence current in per unit of machine rated current and t is time in seconds.

The unbalance limits should always be agreed with the power utilities. For example, in recent railroad electrification projects in the USA, the following limits were used:

- 115 kV system: Voltage unbalance with all systems in: 2%
  Voltage unbalance for outage condition: 2%
- 230 kV system: Voltage unbalance: 3%
  Current unbalance: 5%

The voltage and current unbalance on the utility system can be reduced by allocating power feeding to alternate phases of the supply system at successive substations.

6.4.4 Harmonic Distortion and Resonance

Thyristor control equipment on board locomotives cause harmonic current flows which cause additional voltage drops in the system, distortion of utility system voltages, and increase in heating of rotating plant and capacitors. Harmonic currents in the catenary system may also induce noise in control, telecommunication, and signal circuits. In systems with appreciable susceptance, a particular harmonic may coincide with the natural frequency of the system and cause system resonance which will in turn produce additional distorted currents and voltages.

IEEE Standard 519 recommends the maximum permissible limits for voltage distortion. The distortion limits must be agreed with the power utilities. For example, in recent railroad electrification projects in the USA, the following limits were used:

- 115 kV system: Single harmonic voltage distortion: 1.5%
  Total harmonic voltage distortion: 2.5%
- 230 kV system: Single harmonic voltage distortion: 1%
  Total harmonic voltage distortion: 3%

In the event that the harmonic distortion exceeds the maximum agreed limits, the current and voltage distortion can be controlled by application of on-board or wayside filters.

6.4.5 Voltage Flicker

Rapidly changing traction currents may cause voltage dips which can result in objectionable light flicker. The measure of flicker acceptable by utilities depends on the magnitude and number of the voltage dips. Some utilities allow a maximum level of fluctuation on the worst phase as given in IEEE Standard 141 by the border line of visibility of flicker curve, while other utilities adopt their own standards. It is recommended that a voltage flicker analysis be undertaken for each considered substation supply point and that the results be coordinated with the utility companies.

6.4.6 Power Factor

It is recommended that the power factor be calculated at each substation as a part of the Load Flow and Power Demand Studies and that the results be compared with the utility company limits on leading and lagging power factor. Power factor control can be achieved by the use of on-board equipment on the locomotives/MUs or by the use of wayside equipment. The on-board control can be accomplished by capacitors or by special design of the propulsion equipment, refer to Part 8 of the Manual. Wayside control can use either switched or un-switched capacitors.

The decision whether or not to control the power factor is one of economics. Power utilities usually charge a low power factor penalty which represents the utility cost of corrective measures. When the cost to the railroad to improve the power factor is lower than the present value of the penalty charge for the years ahead, railroad corrective measures should be taken and power factor correcting capacitors installed. This decision can only be made when the rate structure of each power utility with respect to the railroad is established.

6.5 INTERFERENCE STUDIES

Equipment along the electrified railroad right-of-way such as signaling and communication circuits can be subjected to electrical interference from traction and power utility systems. Other facilities, such as metallic fences and roofs, may also be affected.

Interference may give rise to extraneous voltages in electrical circuits or lineside metal work, which must be maintained within acceptable limits to ensure safety of personnel and the public, to prevent damage to equipment, and to maintain satisfactory operation of railroad and utility systems. Therefore, it is recommended that detailed interference studies be performed to evaluate the magnitudes of disturbing effects, assess their acceptability with reference to industry guidelines, and recommend mitigating measures, equipment and systems as necessary.

The International Telegraph and Telephone Consultative Committee (CCITT) Directives, Volumes I-IX, provide comprehensive surveys of various aspects concerning evaluation of interference effects and protection against their harmful consequences. The CCITT Directives are recommended for interference studies concerned with railroad electrification.

6.5.1 Electrostatic Induction

Electrostatic induction (ESI) is caused by capacitive coupling. It is recommended that electrostatic induction be considered from the standpoint of safety. The CCITT Directives consider the relevant factors and recommend a method of calculation of critical spacing between disturbing and disturbed conductors. For conductors outside the critical distance, electrostatic induction can be ignored.

6.5.2 Electromagnetic Induction

Electromagnetic induction (EMI) is caused by inductive coupling. For normal conditions, IEEE has proposed a maximum induced voltage level of 50 Volts but, when special precautions are taken, 150 Volts. The CCITT Directives give 60 Volts as a normal maximum safe value and 150 Volts with special precautions. The maximum allowable longitudinal induced voltage during a disturbing system fault is recommended by the CCITT Directives as 430 Volts for standard lines and 650 Volts for high reliability lines.

In order to maintain satisfactory performance of communication equipment, the combined effect of currents at all harmonic frequencies is recommended to be less than 20 dBrnC, i.e. 20 dB above reference noise C-message weighted.

6.5.3 Interference Mitigation

ESI and EMI interference problems identified during detailed engineering can be reduced by mitigation at the source of interference, by protecting the disturbed conductors, or by a combination of both.

The following equipment is recommended for interference mitigation at the source:

- Booster transformers—return current is forced to flow in the rails or in a parallel conductor, the interference effects of which tend to cancel the disturbing effects of currents in the catenary wires.
- Neutralizing wire—the interference effects of induced currents in a conductor placed adjacent to the disturbed cable tends to cancel the disturbing effects of currents in the catenary wires.
- Autotransformer system—opposing currents flowing in the feeder conductors tend to cancel the disturbing effects of currents flowing in the catenary wires.

Protection of the disturbed conductors can also be achieved by application of:

- Grounding—effective in reducing ESI and EMI
- Cable shielding—effective in reducing EMI
- Isolating transformers—recommended for EMI reduction for a small number of circuits
- Protectors—recommended for reduction of overvoltages
- Drainage to ground—recommended for reduction of EMI
- Neutralizing transformers—recommended for reduction of EMI.

With regard to mitigation of noise in communications circuits, the following methods are recommended:

- Avoiding close parallel construction of telecommunication circuits and catenary circuits
- Assuring Conductor transposition (interchange of conductor positions between successive sections) to compensate for unequal conductor spacing over a length of exposure
- Using cables with grounded steel tape and aluminum shielding conductors
- Improving the signal-to-noise ratio by increasing the signal strength.

Fiber optic circuits are immune to electrical interference and have been applied with success to railroad routes with electrified operation. Fiber optic technology is recommended for voice and data communication.

6.6 SYSTEM DESIGN

Substation equipment is subjected to the full impact of the traction power demand including current fluctuation, presence of harmonics and low power factor. The equipment is also subjected to a relatively high occurrence of catenary system faults which have a significant effect on equipment design.

6.6.1 General

The issues described in the following sections should be established for each item of substation equipment.

6.6.1.1 Service Conditions

It is recommended that the following service conditions be identified in the equipment specifications:

a. Maximum and minimum ambient temperature

b. Maximum and minimum relative humidity

c. Altitude above sea level

d. Existence of any corrosive atmospheres, such as salt spray

e. Indoor or outdoor installation requirement

f. Seismic levels or vibrations from passing trains

g. Short circuit duty at utility substations and line tapping points

h. Short circuit duty at the traction transformer secondary windings

i. Harmonic frequency spectrum magnitude.

6.6.1.2 Standards

All traction power substation equipment is recommended to be designed, built and tested in accordance with specifications prepared specifically for each project. All specifications should be based on relevant ANSI, ASME, IEEE, UL and NEMA standards. Local standards and codes should be verified for applicability.

6.6.1.3 Equipment Design

Traction power equipment is generally located in unmanned substations and switching stations and, therefore, is recommended to be simple and reliable. The equipment should be designed considering ease of access for testing and maintenance.

6.6.1.4 Basic Impulse Insulation Level

An insulation coordination study is recommended to be performed for all voltage levels in the traction power substations, so that suitable Basic Insulation Level (BIL) and appropriate surge arresters can be selected. The primary voltage equipment BIL, applicable to HV circuit breakers, HV disconnect switches and transformer primary windings, must be fully coordinated with the utility system BIL. The low (secondary) voltage system BIL, applicable to transformer secondary windings and switchgear, is recommended to be as shown in the following table:

Electrification Voltage (kV)	Equipment BIL (kV)
12.5	150
25	250
50	450

6.6.1.5 Tests

Comprehensive tests should be specified for all substation equipment including design, production and installation verification tests in accordance with the relevant ANSI and NEMA standards. When large numbers of equipment are being purchased under the same contract, the buyer should reserve the right to repeat the basic acceptance tests on a random sample of the batch in order to maintain quality control.

6.6.1.6 Spare Parts, Special Tools and Test Equipment

All traction power supply equipment is recommended to be ordered with a full complement of spare parts, special tools and test equipment in sufficient quantity to last two years after equipment acceptance. The procurement documents should request guaranteed cost of additional equipment for an order executed within the two years.

6.6.1.7 Documentation

Each manufacturer should be required by specification to furnish a comprehensive set of documentation with the delivered equipment. This documentation is recommended to include product data, fully dimensioned drawings including weights and erection details, preventive maintenance, corrective maintenance and heavy repair manuals, test procedures and test results.

6.6.2 High (Primary) Voltage Circuit Breakers, Disconnect Switches and Protection

Selection of high (primary) voltage circuit breakers, disconnect switches and protective equipment is governed by the circuit voltage level, short circuit fault level, and protection philosophy of the particular electrical power supply utility.

6.6.3 Traction Power Transformers

Consideration should be given to specifying traction power transformers for a particular project with the same characteristics to standardize design and maintenance, and permit equipment interchangeability.

6.6.3.1 Continuous and Overload Current Ratings

Substation transformers should be rated on the basis of the Load Flow Study results; refer to Section 6.3. Each transformer must be rated to supply continuously its own load under normal operating conditions, together with the additional load of the adjacent electrical section under a transformer or substation outage. Because of the traction load fluctuation, it is recommended that the transformers should be specified to supply the rated power continuously with a superimposed overload cycle equal to 150% of continuous rating for 2 hours and 300% of continuous rating for 5 minutes without significant reduction of service life expectancy.

It is recommended that transformers include spare capacity for future increases in train sizes or number of trains in operation. The loading corresponding to the projected traffic density should be used to derive the transformer self-cooled rating (OA) with forced air-cooled rating (FA) and forced air-forced liquid-cooled rating (FOA) reserved for future service growth beyond the projection.

6.6.3.2 Temperature Rise

It is recommended that the transformer winding temperature rise above ambient temperature, based on its continuous rating, should not be permitted to exceed 65°C using resistance measurements and the winding hottest-spot temperature rise should not be permitted to exceed 80°C.

6.6.3.3 Harmonics

Modern electric traction vehicles can generate significant levels of frequency harmonics, refer to Part 8 of this Chapter. Harmonic currents increase the total RMS current loading of the transformer and therefore produce additional heating. Coordination with the vehicle design team is recommended especially if on-board filtering equipment is being considered. It is recommended that the transformer specifications include the projected RMS current values of all harmonics expressed as a percentage of fundamental frequency. As a minimum, harmonics from the third through the twenty-first harmonic should be included.

6.6.3.4 Impedance Ratings

The selected value of impedance must be low enough to avoid excessive voltage drop in order to obtain long substation feeding distances. The impedance should not be too low, however, as this would affect the economy of transformer and low voltage switchgear design. Judgment based on engineering and economic factors is required to obtain the optimum value.

6.6.3.5 Core and Windings

The fluctuating load currents and relatively high incidence of heavy fault currents produce pulsating forces and mechanical stresses in the transformer windings. These forces and stresses may cause axial and radial movement of the coils and eventual transformer failure. It is recommended that the specifications include the requirement for augmented mechanical strength of the transformer core and include an internal bracing system for windings. Winding and tap connections should be located to minimize their movement and damage.

6.6.3.6 Voltage Ratios and Tap Changers

It is recommended that the no load voltage ratio at a normal tap position be that shown in Section 3.4 of this chapter for the "normal upper voltage limit." It is further recommended that 8 no load taps in 2.5% increments be provided to permit adjustments for utility supply variations from -10% to +10% of nominal voltage.

6.6.3.7 Oil Preservation and Pressurization System

It is recommended that each transformer be equipped with oil expansion tanks and an inert gas pressure system along with appropriate gauges, alarms and safety valves. Removable radiators are recommended to facilitate maintenance. The large volume (several hundred gallons) of cooling oil in the transformer tank and radiators creates a large heat sink that can absorb significant temperature increases due to overloads under cyclic loading without any adverse effects on the transformer. The transformers should be installed with appropriate oil containment provisions to minimize environmental damage in the event of a rupture of the tanks or radiators.

6.6.3.8 Transformer Protection

Each traction power transformer is recommended to be equipped with phase and ground fault overcurrent relays and differential relays. Two stage winding temperature relay alarms should be provided, which should be designed to operate cooling fans and provide an alarm at lower excess temperature levels, and open the low voltage circuit breakers at higher excess temperatures. A two-stage sudden pressure relay for internal transformer faults should initiate an alarm for gas accumulation and trip out the transformer for an oil surge.

6.6.3.9 Noise Level

The specifications should include maximum exterior noise levels, in accordance with IEEE/ANSI Standards, if the transformer is to be located in a populated area.

6.6.3.10 Acceptance Tests

Short circuit tests should be considered an essential part of the acceptance procedure, due to the operating environment of the transformer. Tests should be run for each primary voltage type and each rating of the transformer.

6.6.4 Low (Secondary) Voltage Switchgear

The catenary system is susceptible to frequent short circuit faults and, therefore, switchgear with vacuum or sulfur hexafluoride (SF6) circuit breakers is recommended. The circuit breakers should be capable of several hundred operations at short circuit current levels and several thousand operations at rated current levels before maintenance.

6.6.4.1 Switchgear Type

Whenever voltage rating permits, metal-clad switchgear assemblies with horizontal draw-out circuit breakers are recommended. The switchgear should be located in metal or brick housings and installed in dead-front, floor-mounted, free-standing cubicles. Indoor, fixed, metal-enclosed switchgear or outdoor circuit breakers are recommended alternatives to the metal-clad, draw-out circuit breaker type switchgear.

6.6.4.2 Ratings

Switchgear and circuit breakers are recommended to be rated on a symmetrical current basis, refer to the IEEE/ANSI C37 series of standards. The continuous and overload ratings of feeder switchgear should be compatible with the overhead conductor ampacity and the traction transformer rating. The incoming and bus-tie circuit breakers are recommended to be rated at a higher continuous current rating than the feeder circuit breakers.

The switchgear must be able to carry the short circuit current for sufficient time to enable the protective relaying to operate. Once the circuit breaker trip command is issued, current interruption should be fast without restrike due to transient voltage recovery.

6.6.4.3 Catenary Protection

The catenary system can experience high peak load currents and low fault currents which can be comparable in magnitude. This precludes the use of overcurrent type protection, as overcurrent relaying cannot distinguish between the high load and low fault currents.

The most feasible solution for catenary protection is the use of distance relaying. This form of protection is comparatively simple to apply, is of high speed class, and provides primary and back-up facilities inherent in a single scheme. The distance relay measures impedance along the protected line and is arranged to operate for faults between the relay location and a selected point. The length of line is usually divided into three protection zones, thus enabling time discrimination for faults in different line sections, refer to Figure 6.3. In order to accelerate the fault clearance, a transfer trip of each remote circuit breaker is recommended.

A high proportion of catenary faults will clear once the circuit breaker is opened and the air in the fault location is de-ionized. Depending on the railroad operating practices, use of an auto-reclosing operating device is recommended. Such a relay will reclose the circuit breaker after a interval of 3 to 15 seconds, if not manually overridden. In the case of persistent faults, the circuit breaker will latch out on the second or third opening.

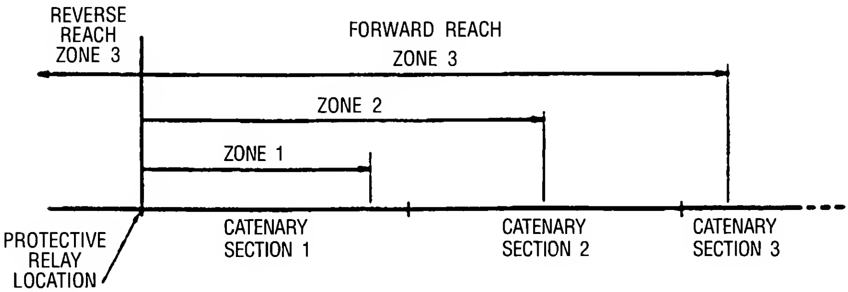
In order to prevent the distance relay operating under train accelerating current, it is recommended that the relay be specified with independently adjustable line resistance and reactance settings. Also, the distance relay can be used in combination with a so called "load blinder" relay, which will preclude circuit breaker tripping during high overload currents.

In addition to distance relaying, consideration should be given to thermal overload protection which prevents the system conductors from overheating and possibly annealing. In special circumstances, where sufficiently high short circuit currents are not available to clear remote faults, a transfer trip protective scheme using pilot wire or fiber optic communications can be considered.

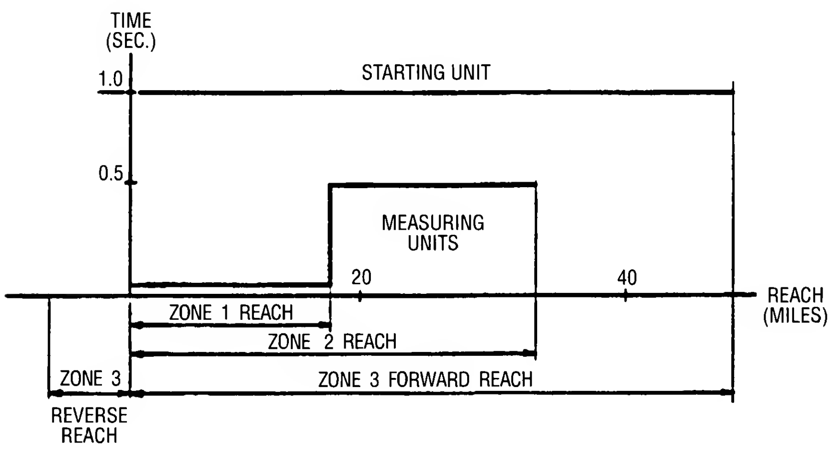
Catenary system protection must be immune to system harmonics and must ensure full discrimination of protective devices. The protection must also provide a complete back-up in the case of breaker or relay failure and be inoperative under in-rush of magnetizing current to on-board locomotive transformers.

6.6.5 Power Factor Correction Capacitors and Harmonic Frequency Filters

Design of equipment such as power factor correction capacitors and harmonic frequency filters should take into consideration the presence of voltages and currents at harmonic frequencies, caused



DISTANCE-REACH CHARACTERISTIC



TIME-REACH CHARACTERISTIC

Figure 6.3. Characteristics of Distance Relay.

not only by traction loads but also harmonics that may be already present on the utility system. With increased frequency of harmonics the equipment impedance decreases and the capacitors may become overloaded and overheated.

6.6.6 Traction Current Return System

Traction current is returned from the locomotives and MU cars to the substations via rails, ground wires, ground, and neutralizing wires when installed. In order to enable both rails to carry the return current and to maintain the double rail signaling track circuits commonly used by North American railroads, any existing dc track circuits must be changed to ac track circuits, and impedance bonds must be installed at signal section block points.

In order to minimize the impedance of the traction current return path, it is beneficial in multi-track areas to have the impedance bond midpoints connected by cross bonds. To minimize rail voltage rise, selected cross bonds should be grounded along the system ROW and connected to substation and switching station grounds. The cross bond grounding must be coordinated with the signaling system design, refer to Part 5 of this Chapter.

6.6.7 Signal Power Generating System

The signal power supply system can be supplied by the local power utility or from the traction power system substations and switching stations. Trackside signal power supply points usually include a motor/generator set or a solid state converter and associated transformers, control and protective equipment.

The system specifications should include limits on frequency and voltage variations. In order to maintain continuous power supply, back-up generators or converters should be provided for a trackside system with automatic power transfer equipment enabling transfer from main to back-up supply during emergencies. For direct utility supply, it is recommended that dual supply lines should be installed with automatic transfer equipment. Depending on the signaling system, the restoration of signal power is required to be achieved in a specified time to avoid interfering with the safety of normal train operations.

6.6.8 Supervisory Control and Data Acquisition System (SCADA)

The use of a computer-based SCADA system is recommended for monitoring and control of the unmanned traction power supply, signaling power supply and system sectioning facilities. As a minimum the system should incorporate the following features:

- Remote control of all circuit breakers, motor-operated disconnect switches and lockout relays
- Status indication of all circuit breakers, disconnect switches and grounding switches
- Status indication of protective relaying, ac auxiliary power equipment and dc auxiliary power equipment including the station battery and battery charger
- Status of communication system
- Enable/disable automatic reclosing of circuit breakers
- Metering of voltages, currents and power factor
- Maximum demand prediction
- Recording of maintenance clearance permits and maintenance status
- Work permit, power removal and out of service equipment tagging
- Catenary power removal coordination with railroad operations and track blocking
- Annunciation of circuit breaker tripping and low substation voltages

- Annunciation of intruder and smoke/fire alarms.
- Sequence-of-events recording to 1 second accuracy
- Voice communication

It is recommended that the SCADA system personnel should interface regularly with centralized train control (CTC) dispatchers to advise them regarding energized and deenergized status of catenaries. The train dispatchers, in turn, should have input to the coordination of traction power system maintenance.

Depending on the system size, it is recommended that the SCADA system be equipped with one or more color visual display units which may be supplemented with modular or rear projection screens for large systems. It is also recommended that the selection and de-selection of equipment and control command transmittal be performed from the computer keyboards. In order to facilitate SCADA system maintenance, software changes and avoid disruption of service due to failures, duplication of the SCADA system is recommended at each railroad control center.

6.7 UTILITY METERING

It is recommended that the electrification system owners begin early discussions with the power utilities to negotiate the most advantageous electrical rate structure and identify any possible connection costs.

6.7.1 Typical Rate Structure

A typical rate structure for provision of electrical power to the railroad may consist of the following:

- Energy charge—includes charge for energy consumed over a billing period of time
- Fuel cost adjustment—includes adjustment for fuel cost variation
- Demand charge—covers the utility generation and transmission costs
- Dedicated utility facility cos— covers major connection costs
- Miscellaneous charges—may include penalties for low power factor and excessive phase unbalance
- Discounts—may include high voltage service and off-peak usage discounts.

6.7.2 Location of Metering Equipment

The metering equipment can be located on the substation high (primary) voltage side or at the traction power transformer secondary. The primary voltage metering requires more expensive HV potential and current transformers, while the low (secondary) voltage metering is more complex as it must compensate for transformer losses. Overall, the secondary voltage metering system is less expensive and therefore is recommended whenever permitted by the utilities.

6.7.3 Billing Concepts

Each power utility may have a different rate structure and the tariff applied to a railroad electrification supply has to be negotiated. It is recommended that the billing concepts presented in this section be included in such negotiations.

6.7.3.1 Conjunctive Billing

Coincidental or conjunctive billing is applicable when several substations are supplied by the same utility. Due to the fact that the train load moves the demand from substation to substation, peak demands on individual substations are unlikely to occur simultaneously. Therefore, coincident power demand measured at several substations during the maximum demand period is likely to be lower than the sum of maximum demands as measured individually at each substation. Considerable billing savings can be realized by totaling the power demands of several substations.

6.7.3.2 Time-of-Day and Time-of-Year Pricing

These pricing concepts are designed to charge a higher rate during peak-load periods and lower rates at other times.

Proposed 1996 Revisions to Portfolio of Trackwork Plans

AREA Specifications for Special Trackwork

Plan. No. 100-92, Page 9, Specification M11. Bolts and Nuts. Replace current article with the following.

M11. BOLTS AND NUTS

M11.1 Material Covered

M11.1.1

Heat treated steel track bolts and nuts for use in standard rail joints, compromise joints, and switch heel joints.

M11.1.2

Heat treated steel bolts and medium carbon steel nuts for use in frogs, crossings, and guard rails of special trackwork.

M11.1.3

Heat treated steel bolts and medium carbon steel nuts for application other than those defined in M11.1.1 and M11.1.2.

M11.2 Manufacture

M11.2.1

Heat treated steel track bolts and nuts as defined in M11.1.1 shall be produced in conformance with the latest issue of ASTM-A183 Standard Specification for Carbon Steel Track Bolts and Nuts.

M11.2.2

The heat treated steel track bolts and nuts (M11.1.1) shall be in accordance with the design and dimensions specified by the purchaser in the order or contract subject to the permissible tolerance variations defined in the specifications.

M11.2.3

Heat treated steel bolts as defined in M11.1.2 and M11.1.3 shall be produced to a Grade 5 designation in conformance with the chemical requirements, mechanical requirements, and in general as far as applicable to the latest issue of The Society of Automotive Engineers Specification SAE J429 for Mechanical and Material Requirements for Externally Threaded Fasteners.

M11.2.4

Where specified by the purchaser, heat treated steel bolts as defined in M11.1.2 and M11.1.3 shall be produced to a Grade 8 designation in conformance with the chemical requirements, mechanical requirements and in general as far as applicable to the latest issue of The Society of Automotive Engineers Specification SAE J429 for Mechanical and Material Requirements for Externally Threaded Fasteners.

M11.2.5

Medium carbon steel nuts as defined in M11.1.2 and M11.1.3 shall be produced to a Grade 5 designation in conformance with the chemical requirements (except minimum carbon content to be 0.40%), mechanical requirements, and in general as far as applicable to the latest issue of The Society of Automotive Engineers Specification SAE J995 for Mechanical and Material Requirements for Steel Nuts.

M11.2.6

Where specified by the purchaser, medium carbon heat treated steel nuts as defined in M11.1.2 and M11.1.3 shall be produced to a Grade 8 designation in conformance with the chemical requirements, mechanical requirements and in general as far as applicable to the latest issue of The Society of Automotive Engineers Specification SAE J995 for Mechanical and Material Requirements for Steel Nuts.

M11.2.7

The grade of the nut shall correspond to the grade of the bolt.

M11.3 Dimensions and Other Physical Attributes

The heat treated bolts and medium carbon steel nuts, other than track bolts, shall conform to the following details.

M11.3.1 Diameter of Bolts

M11.3.1.1

Main bolts or bolts through the body and fillers of frogs, guard rails and crossings of special trackwork and applicable nuts (M11.1.2) shall be of the following diameters:

1⅜" (34.9 mm) for rail sections heavier than 110 lb. per yard (54.6 kg/m).

1¼" (31.8 mm) for rail sections 110 lb. per yard (54.6 kg/m) or lighter, down to and including rails having 3" (76.2 mm) fishing height.

1⅛" (28.6) for rail sections having less than 3" (76.2 mm) fishing height.

M11.3.1.2

The diameter of other bolts and applicable nuts (M11.1.3) shall be as specified in the detail plans.

M11.3.2 Bolts

The dimensional data for bolts for special trackwork (M11.1.2) and (M11.1.3) shall conform to the latest issue of the American National Standard Institute Specification ANSI B18.2.1 for square bolts except where otherwise shown on detail plans.

M11.3.3 Nuts

The dimensional data for nuts shall conform to the latest issue of the American National Standard Institute Specification ANSI B18.2.2 for heavy square nuts.

M11.3.4 Bolt and Nut Threads

M11.3.4.1

The bolt and nut threads shall conform to the latest issue of the American National Standard Institute, Unified Screw Thread Requirements, ANSI B1.1, Course Thread series, UNC with tolerances and allowance in accordance with Class 2A for external threads (bolts), and Class 2B for internal threads (nuts).

M11.3.5 Bolt Body

M11.3.5.1

The bolts shall have unthreaded shanks within the limits of the latest issue of the American National Standard Institute, Square Bolt Requirements, ANSI B18.2.1, but not less than the minimum major diameter of the threads.

M11.3.6 Bolt Fitting

M11.3.6.1

Bolts shall be equipped as shown on the detail plans with head locks, spring washers and flat or beveled washers to provide proper bearing and to permit tightening of the nuts with a standard track wrench. Bearing shall be as defined in the American National Standard Institute Specification ANSI B18.2.1 for square bolts.

M11.4 Identification Symbols

M11.4.1

Heat treated steel bolts, as defined in M11.1.2 and M11.1.3, shall be identified by identification marking symbols on the top of the head of the bolts. Identification marking symbols shall be as per the appropriate SAE Grade Markings for Steel Bolts.

M11.4.2

Medium carbon steel nuts as defined in M11.1.2 and M11.1.3, shall be identified by identification markings as to manufacturer and grade in conformance with the latest issue of the American National Standard Institute Specification ANSI B18.2.2 for Heavy Square Nuts.

M11.5 Workmanship

M11.5.1

All bolts and nuts shall be free of imperfections that affect the serviceability.

Plan No. 1001-55. Replace existing Table of Data for Tee Rail Sections with the following table:

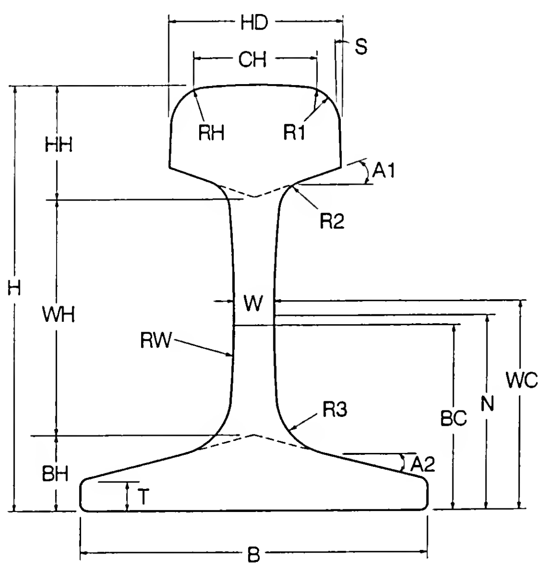
American Railway Engineering Association
Rail Sections

in use since about 1900

Prepared by Committee 18: Light Density and Short Line Railways

Key to Rail Section measurements

A1 Bottom Head Angle
A2 Base Angle
B Base Width
BC Bolt Center Line
BH Base Height
CH Head Radius Width
H Rail Height
HD Head Width
HH Head Height
N Neutral Axis
RH Head Radius
RW Web Radius
R1 Gage Corner Radius
R2 Top Fillet Radius
R3 Bottom Fillet Radius
S Head Side Slope*
T Base Edge Thickness
W Web Thickness
WC Web Center Line
WH Web Height



*On Head Free sections, the bottom of the head is narrower than the top, thus the side slope in the opposite direction from standard sections.

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R
2																		
3			Manufacturer and Section Number														Rail	Base
4	Wt. / Yd.					Old		Tennessee			Colorado			Maryl.			Height	Width
5	lbs.	Type	U.S. Steel	Bethlehem	Illinois	Illinois	Carnegie	TC & I	Lackawanna	Midvale	CF & I	Inland	Cambria	Penn.	Dominion	Algoma	H	B
6	8	ARA-B			841		841	841			81						1 9/16	1 9/16
7		ASCE			840	801	840	840									1 9/16	1 9/16
8	10	ASCE			1040		1040	1040									1 3/4	1 3/4
9		????									101						1 9/16	1 11/16
10	12	ARA-B			1230		1230	1230			122						2 3/64	1 3/4
11		ASCE		12-AS	1240	1201	1240	1240	120	580							2	2
12		PA STL. CO. - MD STL. CO.												66			2	2
13	14	ARA-B			1430		1430	1430			143						2 3/64	1 27/32
14		ASCE					1440										2 1/16	2 1/16
15	16	ARA-B			1630		1630	1630			162						2 7/16	2
16		ASCE		16-AS	1640	1601	1640	1640	160	579							2 3/8	2 3/8
17		PA STL. CO. - MD STL. CO.												48			2 11/32	2 11/32
18	18	ARA-B			1830		1830	1830			182						2 7/16	2 5/64
19	20	ARA-B			2030		2030	2030			203						2 43/64	2 1/4
20		ASCE		20-AS	2040	2001	2040	2040	200	578							2 5/8	2 5/8
21		PA STL. CO. - MD STL. CO.												47			2 1/2	2 1/2
22	25	ARA-B			2530		2530	2530			252						2 13/16	2 1/2
23		ASCE		25-AS	2540	2502	2540	2540	250	577				264			2 3/4	2 3/4
24		PA STL. CO. - MD STL. CO.												45			2 3/4	2 21/32
25	30	ARA-B			3030		3030	3030			303						3 11/64	2 3/4
26		ASCE												269			3	3
27		ASCE		30-AS	3040	3002	3040	3040	300	576	301						3 1/8	3 1/8
28		PA STL. CO. - MD STL. CO.												44			3	3 5/16
29	35	ARA-B			3530		3530	3530			352						3 23/64	3
30		ASCE												265			3 1/4	3 1/4
31		ASCE		35-AS	3540	3502	3540	3540	350	575	351						3 5/16	3 5/16
32		PA STL. CO. - MD STL. CO.												43			3 1/16	3 3/8
33	40	ARA-B			4030		4030	4030			403						3 35/64	3 3/16
34		ASCE		40-AS	4040	4004	4040	4040	400	545	401			138			3 1/2	3 1/2
35		PA STL. CO. - MD STL. CO.												32			3 1/2	3 1/2
36	45	ARA-B			4530		4530	4530			452						3 47/64	3 3/8
37		ASCE		45-AS	4540	4504	4540	4540	450	549	451			266			3 11/16	3 11/16
38		PA STL. CO. - MD STL. CO.												36			3 5/8	3 5/8

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	S	T	U	V	W	X	Y	Z	AA	A8	AC	AD	AE	AF
2								Base			Gage	Top	Bottom	Head	Head	
3			Head	Web	Head	Web	Base	Edge	Head	Web	Corner	Fillet	Fillet	Side	Bottom	Base
4	Wt. / Yd.		Width	Thickness	Height	Height	Height	Thickness	Radius	Radius	Radius	Radius	Radius	Slope	Angle	Angle
5	lbs.	Type	HD	W	HH	WH	BH	T	RH	RW	R1	R2	R3	S	A1	A2
6	8	ARA-B	23/32	5/32	7/16	13/16	5/16		8	12				VERT.	13 deg.	13 deg.
7		ASCE	13/16	5/32	15/32	13/16	9/32		12	12	5/32	3/16	3/16	VERT.	13 deg.	13 deg.
8	10	ASCE	15/16	3/16	33/64	15/16	19/64		12	12	5/32	3/16	3/16	VERT.	13 deg.	13 deg.
9		????	29/32	9/32	7/16	13/16	5/16							VERT.	13 deg.	13 deg.
10	12	ARA-B	59/64	3/16	9/16	1 3/32	25/64		12					4 deg.	13 deg.	13 deg.
11		ASCE	1	3/16	9/16	1 3/32	11/32		12	12	5/32	3/16	3/16	VERT.	13 deg.	13 deg.
12		PA STL. CO. - MD STL. CO.	1 1/16	3/16	19/32	1 5/64	21/64	3/32	4 1/2	4 1/2	3/16	5/32	5/32	5 deg.	13 deg.	13 deg.
13	14	ARA-B	1 1/64	9/32	9/16	1 3/32	25/64							4 deg.	13 deg.	13 deg.
14		ASCE	1 1/16	1/4	5/8	1 3/32	11/32		12	12	5/32	3/16	3/16	VERT.	13 deg.	13 deg.
15	16	ARA-B	1 7/64	7/32	41/64	1 23/64	7/16							4 deg.	13 deg.	13 deg.
16		ASCE	1 11/64	7/32	41/64	1 23/64	3/8		12	12	3/16	3/16	3/16	VERT.	13 deg.	13 deg.
17		PA STL. CO. - MD STL. CO.	1 7/32	7/32	45/64	1 9/32	23/64		4	4 1/2	3/16	5/32	5/32	5 deg.	12 deg.	12 deg.
18	18	ARA-B	1 3/16	19/64	41/64	1 23/64	7/16							4 deg.	13 deg.	13 deg.
19	20	ARA-B	1 17/64	1/4	23/32	1 15/32	31/64							4 deg.	13 deg.	13 deg.
20		ASCE	1 11/32	1/4	23/32	1 15/32	7/16		12	12	1/4	3/16	3/16	VERT.	13 deg.	13 deg.
21		PA STL. CO. - MD STL. CO.	1 7/16	1/4	25/32	1 5/16	13/32	3/32	4 1/2	4 1/2	7/32	3/16	3/16	5 deg.	12 deg.	12 deg.
22	25	ARA-B	1 29/64	9/32	25/32	1 31/64	35/64		12	12	1/4	1/4	1/4	4 deg.	13 deg.	13 deg.
23		ASCE	1 1/2	19/64	25/32	1 31/64	31/64		12	12	1/4	1/4	1/4	VERT.	13 deg.	13 deg.
24		PA STL. CO. - MD STL. CO.	1 9/16	19/64	29/32	1 13/32	7/16	7/64	7	7	7/32	7/32	7/32	5 deg.	12 deg.	12 deg.
25	30	ARA-B	1 19/32	5/16	7/8	1 23/32	37/64		12	12	5/16	1/4	1/4	4 deg.	13 deg.	13 deg.
26		ASCE	1 5/8	21/64	55/64	1 39/64	17/32		12	12	1/4	1/4	1/4	VERT.	13 deg.	13 deg.
27		ASCE	1 11/16	21/64	7/8	1 23/32	17/32		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
28		PA STL. CO. - MD STL. CO.	1 45/64	11/32	15/16	1 9/16	1/2	1/8	7 1/2	9	1/4	1/4	1/4	5 deg.	12 deg.	12 deg.
29	35	ARA-B	1 23/32	11/32	61/64	1 25/32	5/8		12	12	5/16	1/4	1/4	4 deg.	13 deg.	13 deg.
30		ASCE	1 3/4	23/64	15/16	1 47/64	37/64		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
31		ASCE	1 3/4	23/64	61/64	1 25/32	37/64		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
32		PA STL. CO. - MD STL. CO.	1 7/8	3/8	1	1 35/64	33/64	1/8	7 1/2	10	5/16	5/16	5/16	5 deg.	12 deg.	12 deg.
33	40	ARA-B	1 53/64	3/8	1 1/64	1 55/64	43/64		12	12	5/16	1/4	1/4	4 deg.	13 deg.	13 deg.
34		ASCE	1 7/8	25/64	1 1/64	1 55/64	5/8		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
35		PA STL. CO. - MD STL. CO.	1 29/32	25/64	1 1/8	1 55/64	33/64	5/32	5 1/2	11	9/32	5/16	5/16	6 deg.	10 deg.	10 deg.
36	45	ARA-B	1 63/64	13/32	1 1/16	1 31/32	45/64		12	12	5/16	1/4	1/4	4 deg.	13 deg.	13 deg.
37		ASCE	2	27/64	1 1/16	1 31/32	21/32		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
38		PA STL. CO. - MD STL. CO.	1 15/16	13/32	1 9/32	1 51/64	35/64	5/32	4 1/2	9	11/32	1/4	5/16	5 deg.	11 deg.	10 deg.

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Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	AG	AH	AI	AJ	AK	AL	AM	AN	AO	AR
2			Head	Web	Neutral	Bolt						
3			Radius	Center	Axis	Center						
4	Wt. / Yd.		Width	Line	Distance	Line						
5	lbs.	Type	CH	WC	N	BC	REFERENCE					
6	8	ARA-B		23/32	0.70		H ,O					
7		ASCE		11/16	0.75	11/16	D , H , J ,O					
8	10	ASCE		49/64	0.87	49/64	H , J , L ,O					
9		????		23/32			H					
10	12	ARA-B		STR.	0.91		H ,O					
11		ASCE		57/64	0.96	57/64	D , H , J ,O					
12		PA STL. CO. - MD STL. CO.		55/64		55/64	I					
13	14	ARA-B		STR.	0.92		H , L ,O					
14		ASCE		57/64	1.02	57/64	J ,O					
15	16	ARA-B		STR.	1.13		H ,O					
16		ASCE		1 7/128	1.15	1 7/128	D , H , J ,O					
17		PA STL. CO. - MD STL. CO.		1		1	I					
18	18	ARA-B		STR.	1.14		H ,O					
19	20	ARA-B		STR.	1.25		H ,O					
20		ASCE		1 11/64	1.27	1 11/64	D , H , J ,O					
21		PA STL. CO. - MD STL. CO.		1 1/32		1 1/32	I					
22	25	ARA-B		1 37/128	1.30		H ,O					
23		ASCE		1 29/128	1.33	1 29/128	D , H , I , J ,O					
24		PA STL. CO. - MD STL. CO.		1 9/64		1 9/64	I					
25	30	ARA-B		1 7/16	1.50		H ,O					
26		ASCE		1 43/128			I					
27		ASCE		1 25/64	1.52	1 25/64	D , H , J ,O					
28		PA STL. CO. - MD STL. CO.		1 9/32		1 9/32	I					
29	35	ARA-B		1 33/64	1.58		H ,O					
30		ASCE				1 57/128	I					
31		ASCE		1 15/32	1.60	1 15/32	D , H , J ,O					
32		PA STL. CO. - MD STL. CO.		1 9/32		1 9/32	I					
33	40	ARA-B		1 77/128	1.67		H ,O					
34		ASCE		1 71/128	1.68	1 71/128	D , H , I , J ,O					
35		PA STL. CO. - MD STL. CO.		1 7/16		1 7/16	I					
36	45	ARA-B		1 11/16	1.75		H ,O					
37		ASCE		1 41/64	1.78	1 41/64	D , H , I , J ,O					
38		PA STL. CO. - MD STL. CO.		1 15/32		1 15/32	I					

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Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R
2																		
3			Manufacturer and Section Number														Rail	Base
4	Wt. / Yd.					Old		Tennessee			Colorado			Maryl.			Height	Width
5	lbs.	Type	U.S. Steel	Bethlehem	Illinois	Illinois	Carnegie	TC & I	Lackawanna	Midvale	CP & I	Inland	Cambria	Penn.	Dominion	Algoma	H	B
39	48	ARA-B			4830		4830	4830			484						3 47/64	3 29/64
40		ASCE			4840	4803	4840	4840			482						3 11/16	3 49/64
41		M. S. CO.		48-BS													4 1/4	4
42		PA STL. CO. - MD STL. CO.												88			4	4
43	50	ASCE		50-AS	5040	5005	5040	5040	500	542	500			129			3 7/8	3 7/8
44		PA STL. CO. - MD STL. CO.												22			4	4
45	52	RIO GRANDE SO									521						4	4
46	55	ASCE		55-AS	5540	5501	5540	5540	550	537				130			4 1/16	4 1/16
47	56	ASCE		56-AS	5640		5640	5640									4 1/16	4 11/128
48		MISCELL.			5651		5651	5651									4 1/4	3 31/32
49		MISCELL.			5633	5616	5633	5633									4 1/4	4 1/8
50		MISCELL.									562						4 1/4	4 1/8
51		MISCELL.								511							4	3 53/64
52		MISCELL.			5607		5607	5607									4	3 27/32
53		PA STL. CO. - MD STL. CO.												51			4 1/4	4 1/32
54		C.B. & Q.			5608												4 1/16	4 1/16
55	58	PA STL. CO. - MD STL. CO.												7			4 1/4	4 1/4
56		PA STL. CO. - MD STL. CO.												63			4	4
57	60	ARA-A	6020		6020		60RA	602					568			121	4 1/2	4
58		ARA-B	6030		6030		60RB	6030					571				4 3/16	3 11/16
59		ASCE	6040	60AS	6008	6015	60a	600	600	533	603		533	244	6001	105	4 1/4	4 1/4
60		ASCE			6007		60K-R						527				4 1/4	4 1/4
61		ASCE			6013		60M-O		600X				509				VARI	4 1/4
62		ASCE			6012									71	6002	129		
63		ASCE					60P						514					
64		ASCE	6087					6087			4840						4 5/8	4 1/2
65		MISCELL.	6033		6017		60N	6033		503			503				4 1/4	4 13/16
66		MISCELL.	6051		6051	6001	6051	6051									4 1/4	4 1/16
67		PA STL. CO. - MD STL. CO.												6			4 1/4	4 1/4
68		QUEBEC CENTRAL															4 3/8	4 3/8
69	65	ASCE	6540	65AS	6540	6507	6540	6540	650	534	653			236			4 7/16	4 7/16
70		CANADIAN PACIFIC		654	6508		6508	6508	654								4 31/64	4 3/8
71		MISCELL.				6501											4 3/8	4 7/16

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF
2								Base			Gage	Top	Bottom	Head	Head	
3			Head	Web	Head	Web	Base	Edge	Head	Web	Corner	Fillet	Fillet	Side	Bottom	Base
4	Wt. / Yd.		Width	Thickness	Height	Height	Height	Thickness	Radius	Radius	Radius	Radius	Radius	Slope	Angle	Angle
5	lbs.	Type	HD	W	HH	WH	BH	T	RH	RW	R1	R2	R3	S	A1	A2
39	48	ARA-B	2 1/16	31/64	1 1/16	1 31/32	45/64		12	12	5/16	1/4	1/4	4 deg.	13 deg.	13 deg.
40		ASCE	2 5/64	1/2	1 1/16	1 31/32	21/32							VERT.	13 deg.	13 deg.
41		M. S. CO.	1 29/32	3/8	1 1/8	2 29/64	43/64							VERT.	13 deg.	13 deg.
42		PA STL. CO. - MD STL. CO.	1 29/32	3/8	1 3/8	2	5/8	1/8	10	15	1/4	1/4	1/4	VERT.	13 deg.	13 deg.
43	50	ASCE	2 1/8	7/16	1 1/8	2 1/16	11/16		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
44		PA STL. CO. - MD STL. CO.	2 3/32	13/32	1 3/8	1 63/64	41/64	1/8	10	8	7/16	1/4	1/4	5 deg.	13 deg.	13 deg.
45	52	RIO GRANDE SO.	2 1/8	25/64	1 23/64	2	41/64							VERT.	13 deg.	13 deg.
46	55	ASCE	2 1/4	15/32	1 11/64	2 11/64	23/32		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
47	56	ASCE	2 35/128	63/128	1 91/512	2 41/256	371/512							VERT.	13 deg.	13 deg.
48		MISCELL.	2 7/32	13/32	1 7/16	2 1/8	11/16							1/10 to 1	14 deg.	12° 50'
49		MISCELL.	2 1/4	3/8	1 27/64	2 1/8	45/64	1/8	10	8	7/16	1/4	1/4	5 deg.	13 deg.	13 deg.
50		MISCELL.	2 1/4	53/128	1 7/32	2 17/64	49/64							VERT.	13 deg.	13 deg.
51		MISCELL.	2 19/64	29/64	1 51/128	2 59/64	87/128							1/32 to 5/8	12 deg.	12 deg.
52		MISCELL.	2 5/16	15/32	1 13/32	1 29/32	11/16							1/32 to 5/8	12 deg.	12 deg.
53		PA STL. CO. - MD STL. CO.	2 9/32	27/64	1 27/64	2 1/8	45/64	1/8	10	12	7/16	5/16	5/16	6 deg.	13 deg.	13 deg.
54		C.B. & Q.	2 7/32	15/32	1 13/64	2 1/8	47/64		12	11	1/4	3/8	3/8	VERT.	13 deg.	13 deg.
55	58	PA STL. CO. - MD STL. CO.	2 3/16	13/32	1 27/64	2 1/8	45/64	1/8	14	10	1/8	1/4	1/4	3 deg.	13 deg.	13 deg.
56		PA STL. CO. - MD STL. CO.	2 11/32	1/2	1 15/32	1 27/32	11/16	1/4	11	11	7/16	1/4	1/4	3 deg.	15 deg.	12 deg.
57	60	ARA-A	2 1/4	15/32	1 15/64	2 29/64	13/16		14	14	3/8	3/8	3/8	1/16 to 1	1 to 4	1 to 4
58		ARA-B	2 1/8	31/64	1 1/4	2 1/16	7/8		12	12	3/8	5/16	5/16	3 deg.	13 deg.	13 deg.
59		ASCE	2 3/8	31/64	1 7/32	2 17/64	49/64		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
60		ASCE	VARI													
61		ASCE														
62		ASCE														
63		ASCE														
64		ASCE	2 1/2													
65		MISCELL.	2 21/64	29/64	1 55/128	2 7/64	91/128	1/8	10	8	7/16	1/4	1/4	5 deg.	13 deg.	13 deg.
66		MISCELL.	2 5/16	1/2	1 7/16	2 1/8	11/16							1/10 to 1	14 deg.	12° 50'
67		PA STL. CO. - MD STL. CO.	2 3/8	1/2	1 27/64	2 1/8	45/64		10		7/16	1/4	1/4		13 deg.	13 deg.
68		QUEBEC CENTRAL	2 1/4	1/2	1 1/4	2 5/16	13/16		12	12	1/4	1/4	1/4	VERT.	13 deg.	13 deg.
69	65	ASCE	2 13/32	1/2	1 9/32	2 3/8	25/32		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
70		CANADIAN PACIFIC	2 1/4	15/32	1 9/32	2 11/32	55/64							1/32 to 3/4	1 to 4	1 to 4
71		MISCELL.	2 3/8	29/64	1 1/2	2 5/32	23/32							1/16 to 3/4	14° 30'	12° 30'

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	AG	AH	AI	AJ	AK	AL	AM	AN	AO	AR
2			Head	Web	Neutral	Bolt						
3			Radius	Center	Axis	Center						
4	Wt. / Yd.		Width	Line	Distance	Line						
5	lbs.	Type	CH	WC	N	BC	REFERENCE					
39	48	ARA-B		1 11/16			H ,O					
40		ASCE		1 41/64		1 41/64	D , H					
41		M. S. CO.		1 115/128			H					
42		PA STL. CO. - MD STL. CO.		1 5/8		1 5/8	I					
43	50	ASCE		1 23/32	1.88	1 23/32	D , I , J					
44		PA STL. CO. - MD STL. CO.		1 5/8		1 5/8	I					
45	52	RIO GRANDE SO.		1 41/64		1 41/64	D , H					
46	55	ASCE		1 103/128	1.97	1 103/128	D , H , I , J					
47	56	ASCE		1 103/128			H					
48		MISCELL.		1 3/4		1 13/16	D , H					
49		MISCELL.		1 49/64		1 49/64	D , H , J					
50		MISCELL.		1 115/128		1 115/128	D , H					
51		MISCELL.				1 41/64	D					
52		MISCELL.		1 41/64			H					
53		PA STL. CO. - MD STL. CO.		1 49/64		1 49/64	I					
54		C.B. & Q.					N					
55	58	PA STL. CO. - MD STL. CO.		1 49/64		1 49/64	I					
56		PA STL. CO. - MD STL. CO.		1 39/64		1 39/64	I					
57	60	ARA-A		2 17/64	2.13	2 5/128	D , E , F , J ,O					
58		ARA-B		1 29/32	1.95	1 29/32	D , E , F , H , J					
59		ASCE		1 115/128	2.05	1 115/128	D , E , F , I , J					
60		ASCE					F					
61		ASCE					F					
62		ASCE					F					
63		ASCE					F					
64		ASCE					F					
65		MISCELL.		1 49/64		1 49/64	D , F , H , J					
66		MISCELL.		1 3/4		1 3/4	A , D , H					
67		PA STL. CO. - MD STL. CO.		1 49/64		1 49/64	I					
68		QUEBEC CENTRAL					N					
69	65	ASCE		1 31/32	2.14	1 31/32	A , D , E , I , J					
70		CANADIAN PACIFIC		2 1/32		2 1/32	D , H					
71		MISCELL.				1 51/64	D					

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R
2			Manufacturer and Section Number															
3																	Rail	Base
4	Wt. / Yd.					Old		Tennessee			Colorado			Maryl.			Height	Width
5	lbs.	Type	U.S. Steel	Bethlehem	Illinois	Illinois	Carnegie	TC & I	Lackawanna	Midvale	CF & I	Inland	Cambria	Penn.	Dominion	Algoma	H	B
72		MISCELL.				6504											4 1/2	4 1/2
73	66	MISCELL.															4 13/32	4 7/16
74		NORTHERN PACIFIC			6602		6602	6602		547				145			4 17/32	4 1/2
75	67	MISCELL.	6733		6704		6704	6704		515							4 1/2	4 1/2
76		PA STL. CO. - MD STL. CO.												2			4 1/2	4 1/2
77	67.5	RUSSIAN								587							5 3/64	4 21/64
78	68	MISCELL.															4 3/8	4 1/2
79		PA STL. CO. - MD STL. CO.												62			4 1/2	4 1/2
80	70	ARA-A			7020		7020	7020	7031								4 3/4	4 1/4
81		ARA-B		174	7030		7030	7030	7032								4 35/64	4 3/64
82		ASCE	7040	70AS	7040	7010	7040	7040	700	532	701			237			4 5/8	4 5/8
83		BANGOR & AROOSTOOK		70-BA					703					97			4 3/4	4 3/4
84		CHICAGO & ALTON				7002											4 3/8	4
85		PENNSYLVANIA			7033	7005	7033	7033		504				57			4 1/2	4 1/2
86	72	CAN. PAC. (SANDBERG)															4 15/16	4 1/2
87		CHI. & NOR.WESTERN	7250	72NP	7250	7201	7250	7250		581							4 3/4	4 3/4
88		SPOKANE INT'L. RY.							722								4 45/64	4 5/8
89	74	PA STL. CO. - MD STL. CO.												146			4 11/16	4 3/4
90	75	ASCE	7540	75AS	7540	7506	7540	7540	750	529	753			214			4 13/16	4 13/16
91		BOSTON & MAINE		92					752					92			5	5
92		LACKAWANNA		75-C					753								4 11/16	5
93		INT. & GRT. NOR			7551		7551	7551									4 3/4	5
94		MISCELL.															4 3/4	4 1/2
95		MISSOURI PACIFIC	7550	75MP	7550	7512	7550	7550	754	528							4 3/4	4 3/4
96		NAT. RY. MEX.		128													5	5
97		N.Y.C. (DUDLEY)															5	4 3/4
98		PA STL. CO. - MD STL. CO.												87			4 3/4	4 3/4
99		SEABOARD (DUDLEY)		75DY	7522		7522	7522						221			5	5
100		UNION PACIFIC		75-B	7523	7513	7523	7523			754			249			5	5
101		UNION PACIFIC	7524	75SP	7524		7524	7524			757						4 15/16	4 7/16
102	76	PA STL. CO. - MD STL. CO.												216			4 3/4	5
103	77.5	GREAT NORTHERN				77501			775								5	5
104	78	OLD COLONY		78-OC										98			4 3/4	5

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Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF
2 / 3 / 4	Wt. / Yd.		Head Width	Web Thickness	Head Height	Web Height	Base Height	Base Edge Thickness	Head Radius	Web Radius	Gage Corner Radius	Top Fillet Radius	Bottom Fillet Radius	Head Side Slope	Head Bottom Angle	Base Angle
5	lbs.	Type	HD	W	HH	WH	BH	T	RH	RW	R1	R2	R3	S	A1	A2
72		MISCELL.	2 7/16	1/2	1 31/64	2 19/64	23/32							4 deg.	13 deg.	13 deg.
73	66	MISCELL.	2 13/32	15/32	1 17/32	2 1/8	3/4							CURV. 4 1/2 R	13 deg.	13 deg.
74		NORTHERN PACIFIC	2 5/16	17/32	1 27/64	2 11/32	49/64		12	12	1/4	1/4	1/4	VERT.	13 deg.	13 deg.
75	67	MISCELL.	2 13/32	1/2	1 5/8	2 1/8	3/4	1/8	10	8	7/16	1/4	1/4	1/16 to 3/4	13 deg.	13 deg.
76		PA STL. CO. - MD STL. CO.	2 3/8	1/2	1 21/32	2 7/64	47/64	1/8	11	11	11/32	1/4	1/4	5 deg.	14 deg.	13 deg.
77	67.5	RUSSIAN	2 23/64	15/32	1 29/64	2 11/16	29/32							VERT.	1 to 3	1 to 3
78	68	MISCELL.	2 1/2	1/2	1 15/32	2 1/8	25/32							1/16 to 11/16	13 deg.	13 deg.
79		PA STL. CO. - MD STL. CO.	2 15/32	1/2	1 35/64	2 3/16	49/64	3/32	12	8	1/2	1/4	1/4	6 deg.	13 deg.	13 deg.
80	70	ARA-A	2 3/8	1/2	1 11/32	2 1/2	29/32		14	14	3/8	3/8	3/8	1/16 to 1	1 to 4	1 to 4
81		ARA-B	2 3/8	33/64	1 23/64	2 17/64	59/64		12	12	3/8	5/16	5/16	3 deg.	13 deg.	13 deg.
82		ASCE	2 7/16	33/64	1 11/32	2 15/32	13/16		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
83		BANGOR & AROOSTOOK	2 7/16	1/2	1 13/32	2 19/32	3/4	1/8	12	12	1/4	1/4	1/4	VERT.	12 deg.	12 deg.
84		CHICAGO & ALTON	2 35/96	35/64	1 17/24	1 11/12	3/4							1/24 to 5/16	12 deg.	12 deg.
85		PENNSYLVANIA	2 7/16	1/2	1 19/32	2 1/8	25/32	9/64	10	8	7/16	1/4	1/4	4 deg.	13 deg.	13 deg.
86	72	CAN. PAC. (SANDBERG)	2 1/4	1/2	1 5/8	2 25/64	59/64		6	VERT.	1/2	3/8	3/8	VERT.	15 deg.	15 deg.
87		CHI. & NOR.WESTERN	2 3/8	9/16	1 13/32	2 1/2	27/32							VERT.	14 deg.	14 deg.
88		SPOKANE INT'L. RY.	2 7/16	33/64	1 27/64	2 15/32	13/16		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
89	74	PA STL. CO. - MD STL. CO.	2 7/16	9/16	1 3/4	2 3/16	3/4		15	15	3/8	5/16	5/16		17 deg.	13 deg.
90	75	ASCE	2 15/32	17/32	1 27/64	2 35/64	27/32		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
91		BOSTON & MAINE	2 1/2	9/16	1 7/16	2 47/64	53/64		12	STR.	1/4	1/4	1/4	VERT.	13 deg.	13 deg.
92		LACKAWANNA	2 1/2	1/2	1 43/64	2 13/64	13/16		10 1/2	10	7/16	5/16	5/16	1/16 to 1	18 deg.	12° 45'
93		INT. & GRT. NOR	2 1/2	9/16	1 7/16	2 15/32	27/32							VERT.	13 deg.	13 deg.
94		MISCELL.	2 1/2	1/2	1 27/32	2 1/8	25/32							6 deg.	13 deg.	13 deg.
95		MISSOURI PACIFIC	2 9/16	9/16	1 7/16	2 15/32	27/32		12	30	3/8	1/4	1/4	VERT.	13 deg.	13 deg.
96		NAT. RY. MEX.	2 3/4	1/2	1 3/8	2 7/8	3/4							VERT.	12 deg.	12 deg.
97		N.Y.C. (DUDLEY)	2 5/8	17/32	1 3/8	2 3/4	7/8	9/32	14	14	1/4	1/2	5/16	4 deg.	14 deg.	14 deg.
98		PA STL. CO. - MD STL. CO.	2 1/2	9/16	1 1/2	2 7/16	13/16		12	VERT.	1/4	1/4	1/4	VERT.	13 deg.	13 deg.
99		SEABOARD (DUDLEY)	2 9/16	1/2	1 3/8	2 3/4	7/8		14	14	5/16	1/2	5/16	3 deg.	14 deg.	14 deg.
100		· UNION PACIFIC	2 9/16	33/64	1 3/8	2 13/16	13/16		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
101		UNION PACIFIC	2 7/16	33/64	1 3/8	2 5/8	15/16		14	14	3/8	3/8	3/8	1/16 to 1	1 to 4	1 to 4
102	76	PA STL. CO. - MD STL. CO.	2 1/2	1/2	1 11/16	2 1/4	13/16	5/32	20	VERT.	3/8	5/16	5/16		14° 30'	12 deg.
103	77.5	GREAT NORTHERN	2 3/8	5/8	1 11/16	2 1/2	13/16							5 deg.	14 deg.	14 deg.
104	78	OLD COLONY	2 1/2	17/32	1 3/4	2 7/32	25/32		12	12	3/8	7/16	7/16	5 deg.	14 deg.	12° 15'

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	AG	AH	AI	AJ	AK	AL	AM	AN	AO	AR
2			Head	Web	Neutral	Bolt						
3			Radius	Center	Axis	Center						
4	Wt. / Yd.		Width	Line	Distance	Line						
5	lbs.	Type	CH	WC	N	BC	REFERENCE					
72		MISCELL.				1 7/8	D					
73	66	MISCELL.		1 13/16			H					
74		NORTHERN PACIFIC		1 15/16		1 15/16	D , H , I					
75	67	MISCELL.		1 13/16		1 13/16	A , D , J					
76		PA STL. CO. - MD STL. CO.		1 25/32		1 25/32	I					
77	67.5	RUSSIAN				2 1/4	D					
78	68	MISCELL.		1 27/32			H					
79		PA STL. CO. - MD STL. CO.		1 55/64		1 55/64	I					
80	70	ARA-A		2 13/32	2.20	2 5/32	C , E , J					
81		ARA-B		2 7/128	2.16	2 7/128	C , D , E , J					
82		ASCE		2 3/64	2.22	2 3/64	A , C , D , E , I , J					
83		BANGOR & AROOSTOOK		2 3/64		2 3/64	D , H , I					
84		CHICAGO & ALTON				1 17/24	D					
85		PENNSYLVANIA		1 27/32		1 27/32	D , E , H , I					
86	72	CAN. PAC. (SANDBERG)				2 15/128	N					
87		CHI. & NOR.WESTERN		2 3/32		2 3/32	A , D , E					
88		SPOKANE INT'L. RY.		2 3/64	2.25	2 3/64	C					
89	74	PA STL. CO. - MD STL. CO.		1 7/8		1 7/8	I					
90	75	ASCE .		2 15/128	2.30	2 15/128	A , C , D , E , I , J					
91		BOSTON & MAINE		STR.	2.37	2 1/8	C , I					
92		LACKAWANNA		1 117/128	2.24	1 117/128	C , D					
93		INT. & GRT. NOR		2 5/64			E , H					
94		MISCELL.		1 27/32			H					
95		MISSOURI PACIFIC		2 5/64	2.29	2 5/64	A , C , D					
96		NAT. RY. MEX.				2 3/16	D					
97		N.Y.C. (DUDLEY)				2 1/8	N					
98		PA STL. CO. - MD STL. CO.		STR.		2 1/32	I					
99		SEABOARD (DUDLEY)		2 5/8		2 1/4	D , H , I , J					
100		UNION PACIFIC		2 5/8		2 1/4	D , H , I , J					
101		UNION PACIFIC		2 1/2		2 1/4	A , D , J					
102	76	PA STL. CO. - MD STL. CO.		STR.		1 15/16	I					
103	77.5	GREAT NORTHERN				2 1/16	D					
104	78	OLD COLONY		1 57/64			H , I					

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R
2																		
3			Manufacturer and Section Number														Rail	Base
4	Wt. / Yd.					Old		Tennessee			Colorado			Maryl.			Height	Width
5	lbs.	Type	U.S. Steel	Bethlehem	Illinois	Illinois	Carnegie	TC & I	Lackawanna	Midvale	CF & I	Inland	Cambria	Penn.	Dominion	Algoma	H	B
105	79	PA STL. CO. - MD STL. CO.												76			4 3/4	4 3/4
106	79.5	FRICTIONLESS		79.5-C													5 3/16	5 3/16
107	80	ARA-A	8020	80-RA	8020		8020	8020	8031		801						5 1/8	4 5/8
108		ARA-B	8030	80-RB	8030		8030	8030	8032	569	802						4 15/16	4 7/16
109		ASCE	8040	80AS	8040		8040	8040	800	530	800	8040		251			5	5
110		CAN. NOR.		804	8010		8010	8010	804								5	5
111		DUDLEY	8022	80DY	8022		8022	8022						220			5 1/8	5
112		FRICTIONLESS		80-MC-F													5 3/16	5 3/16
113		GREAT NORTHERN				8009			802								5	5
114		HOCKING VALLEY								540							5	4 59/64
115		NEW YORK CENTRAL		220	8022	8008	8022	8022	801	543							5 1/8	5
116	85	ASCE	8540	85AS	8540	8504	8540	8540	850	531	851	8540		235			5 3/16	5 3/16
117		C.B. & Q.	8543	85-CB	8543	8506	8543	8543	855		852						5 3/16	5 3/16
118		CANADIAN PACIFIC	8524	85CP	8524		8524	8524	856			8524					5 1/8	5
119		CANADIAN PACIFIC		85-CP	8524		8524	8524	856			8524					5 1/8	5
120		DENVER & RIO GRANDE									850						5 1/4	5 1/4
121		D. & R.G. / C & S									853						5 3/8	4 7/8
122		GREAT NORTHERN		854	8553	8509	8553	8553	854								5	5
123		MISSOURI PACIFIC	8550	853	8550	8507	8550	8550									5 7/32	5 1/4
124		N.Y.C. & STL. / KCS		85-NK	8521		8521	8521	8531			8521					5 3/8	4 7/8
125		PENNSYLVANIA	8531	85PS	8531	8530	8531	8531	8530	559		8531					5 1/8	4 5/8
126		PENNSYLVANIA		85-PR	8533	8503	8533	8533	852	500				67			5	5
127		SEABOARD (DUDLEY)		85DY	8522		8522	8522	851					261			5 1/4	5
128		SOO LINE	8520		8520		8520	8520									5 3/8	4 7/8
129		WESTERN PACIFIC															5 1/4	5 1/4
130	90	ARA-A	9020	90RA	9020		9020	9020	9031	563	902	9020					5 5/8	5 1/8
131		ARA-B	9030	90RB	9030		9030	9030	9032	561	905	9030					5 17/64	4 49/64
132		ASCE	9040	90AS	9040	9002	9040	9040	900	535		9040		245			5 3/8	5 3/8
133		AT&SF	9021	90SF	9021		9021	9021	9033		903	9021					5 5/8	5 3/16
134		CHI. & NOR.WESTERN	9035	90OM	9035		9035	9035	904								5 17/32	5 3/32
135		DENVER & RIO GRANDE									906						5 1/2	5 1/8
136		FRICTIONLESS			9039		9039	9039									5 5/8	5 1/8
137		FRICTIONLESS			9029		9029	9029									6 3/32	5 1/8

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF
2–4	Wt. / Yd.		Head Width	Web Thickness	Head Height	Web Height	Base Height	Base Edge Thickness	Head Radius	Web Radius	Gage Corner Radius	Top Fillet Radius	Bottom Fillet Radius	Head Side Slope	Head Bottom Angle	Base Angle
5	lbs.	Type	HD	W	HH	WH	BH	T	RH	RW	R1	R2	R3	S	A1	A2
105	79	PA STL. CO. - MD STL. CO.	2 5/8	5/8	1 5/8	2 11/32	25/32	1/8	12	9	1/2	1/4	1/4	6 deg.	13 deg.	13 deg.
106	79.5	FRICTIONLESS	1 15/16	9/16	2 1/32	2 9/32	7/8							4 deg.	13 deg.	13 deg.
107	80	ARA-A	2 1/2	33/64	1 7/16	2 23/32	31/32		14	14	3/8	3/8	3/8	1/16 to 1	1 to 4	1 to 4
108		ARA-B	2 7/16	35/64	1 15/32	2 15/32	1		12	12	3/8	5/16	5/16	3 deg.	13 deg.	13 deg.
109		ASCE	2 1/2	35/64	1 1/2	2 5/8	7/8		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
110		CAN. NOR.	2 9/16	35/64	1 13/32	2 11/16	29/32							1/16 to 1	13 deg.	13 deg.
111		DUDLEY	2 21/32	17/32	1 1/2	2 3/4	7/8		14	14	1/4	1/2	5/16	1/16 to 1	1 to 4	1 to 4
112		FRICTIONLESS	1 15/16	9/16	2 1/32	2 9/32	7/8							4 deg.	13 deg.	13 deg.
113		GREAT NORTHERN	2 13/32	5/8	1 5/8	2 1/2	7/8							5 deg.	14 deg.	14 deg.
114		HOCKING VALLEY	2 31/64	29/64	1 95/128	2 25/64	111/128							4 deg.	13 deg.	13 deg.
115		NEW YORK CENTRAL	2 21/32	17/32	1 1/2	2 3/4	7/8		14	14	5/16	1/2	5/16	1/16 to 1	1 to 4	1 to 4
116	85	ASCE	2 9/16	9/16	1 35/64	2 3/4	57/64		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
117		C.B. & Q.	2 21/32	9/16	1 35/64	2 3/4	57/64							5 deg.	13 deg.	13 deg.
118		CANADIAN PACIFIC	2 1/2	9/16	1 7/16	2 11/16	1	1/4	8	8	1/4	3/8	3/8	1/32 to 1	1 to 4	1 to 4
119		CANADIAN PACIFIC	2 1/2	9/16	1 7/16	2 11/16	1	1/4	8	8	1/4	3/8	3/8	1/32 to 1	1 to 4	1 to 4
120		DENVER & RIO GRANDE	2 1/2	9/16	1 3/4	2 5/8	7/8							4 deg.	13 deg.	13 deg.
121		D. & R.G. / C & S	2 1/2	9/16	1 15/32	2 29/32	1							1/16 to 1	1 to 4	1 to 4
122		GREAT NORTHERN	2 21/32	21/32	1 19/32	2 1/2	29/32							5 deg.	14 deg.	14 deg.
123		MISSOURI PACIFIC	2 15/32	75/128	1 3/4	2 39/64	55/64							2° 30'	13 deg.	13 deg.
124		N.Y.C. & STL. / KCS	2 17/32	17/32	1 29/64	2 15/16	63/64		14	14	3/8	3/8	3/8	1/16 to 1	1 to 4	1 to 4
125		PENNSYLVANIA	2 1/2	17/32	1 21/32	2 15/32	1		10	10	7/16	1/4	1/4	VERT.	15 deg.	13 deg.
126		PENNSYLVANIA	2 9/16	17/32	1 3/4	2 3/8	7/8	5/32	10	8	7/16	1/4	1/4	4 deg.	13 deg.	13 deg.
127		SEABOARD (DUDLEY)	2 11/16	17/32	1 5/8	2 3/4	7/8		14	14	5/16	1/2	5/16	1/16 to 1	1 to 4	1 to 4
128		SOO LINE	2 1/2	9/16	1 15/32	2 29/32	1							3° 34' 35"	14° 2' 11"	14° 2' 11"
129		WESTERN PACIFIC	2 1/2	9/16	1 3/4	2 5/8	7/8		10	VERT.	7/16	5/16	5/16	4 deg.	13 deg.	13 deg.
130	90	ARA-A	2 9/16	9/16	1 15/32	3 5/32	1		14	14	3/8	3/8	3/8	1/16 to 1	1 to 4	1 to 4
131		ARA-B	2 9/16	9/16	1 39/64	2 5/8	1 1/32		12	12	3/8	5/16	5/16	3 deg.	13 deg.	13 deg.
132		ASCE	2 5/8	9/16	1 19/32	2 55/64	59/64		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
133		AT&SF	2 9/16	9/16	1 15/32	3 5/32	1							1/16 to 1	1 to 4	1 to 4
134		CHI. & NOR.WESTERN	2 1/2	1/2	1 17/32	2 31/32	1 1/32		12	12	3/8	5/16	5/16	3/64 to 3/4	13 deg.	13 deg.
135		DENVER & RIO GRANDE	2 9/16	9/16	1 5/8	2 7/8	1							4 deg.	14 deg.	14 deg.
136		FRICTIONLESS	2 1/4	9/16	2	2 5/8	1							1/4 to 1 3/8	13 deg.	13 deg.
137		FRICTIONLESS	1 59/64	9/16	1 15/16	3 5/32	1							1/16 to 1	1 to 4	1 to 4

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	AG	AH	AI	AJ	AK	AL	AM	AN	AO	AR
2			Head	Web	Neutral	Bolt						
3			Radius	Center	Axis	Center						
4	Wt. / Yd.		Width	Line	Distance	Line						
5	lbs.	Type	CH	WC	N	BC	REFERENCE					
105	79	PA STL. CO. - MD STL. CO.				1 15/16	I					
106	79.5	FRICTIONLESS		2 1/64		2 1/64	D , H					
107	80	ARA-A		2 9/16	2.31	2 21/64	C , E , H , J					
108		ARA-B		2 15/64	2.27	2 15/64	C , D , E , H , J					
109		ASCE		2 3/16	2.38	2 3/16	A , C , D , E , I , J					
110		CAN. NOR.		2 1/4		2 1/4	D , H , L					
111		DUDLEY		2 5/8		2 1/4	A , I , J					
112		FRICTIONLESS		2 1/64			H					
113		GREAT NORTHERN				2 1/8	D					
114		HOCKING VALLEY				2 1/16	D					
115		NEW YORK CENTRAL		2 5/8	2.43	2 1/4	C , D					
116	85	ASCE		2 17/64	2.47	2 17/64	A , C , D , E , I , J					
117		C.B. & Q.		2 17/64		2 17/64	A , D , E					
118		CANADIAN PACIFIC		2 11/32		2 11/32	A , D , J					
119		CANADIAN PACIFIC		2 11/32	2.28	2 11/32	C , E					
120		DENVER & RIO GRANDE		STR		2 3/16	A , D					
121		D. & R.G. / C & S				2 29/64	D					
122		GREAT NORTHERN		2 5/32		2 5/32	D , E					
123		MISSOURI PACIFIC		STR		2 21/128	A , D , E , H					
124		N.Y.C. & STL. / KCS		2 23/32	2.43	2 29/64	C , E , J , L					
125		PENNSYLVANIA		2 15/64	2.38	2 15/64	A , C , D , E					
126		PENNSYLVANIA		2 1/16		2 1/16	D , E , I					
127		SEABOARD (DUDLEY)		2 5/8	2.57	2 1/4	C , D , I					
128		SOO LINE		2 23/32		2 29/64	A , D , E , L , M					
129		WESTERN PACIFIC				2 3/16	N					
130	90	ARA-A		2 29/32	2.54	2 37/64	A , C , D , E , J					
131		ARA-B		2 11/32	2.45	2 11/32	A , C , D , E , J					
132		ASCE		2 45/128	2.55	2 45/128	A , C , D , E , I , J					
133		AT&SF		2 29/32		2 37/64	A , C , D , E					
134		CHI. & NOR.WESTERN		2 33/64	2.47	2 23/64	A , D , E ,O					
135		DENVER & RIO GRANDE		2 5/8		2 7/16	A , D					
136		FRICTIONLESS		2 5/16		2 5/16	D , H					
137		FRICTIONLESS		2 37/64		2 37/64	D , H					

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Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R
2																		
3			Manufacturer and Section Number														Rail	Base
4	Wt. / Yd.					Old		Tennessee			Colorado			Maryl.			Height	Width
5	lbs.	Type	U.S. Steel	Bethlehem	Illinois	Illinois	Carnegie	TC & I	Lackawanna	Midvale	CF & I	Inland	Cambria	Penn.	Dominion	Algoma	H	B
138		GREAT NORTHERN	9024	90GH	9024		9024	9024			908	9024					5 3/8	5
139		GREAT NORTHERN			9036		9036										5 3/8	5
140		GREAT NORTHERN		90-GN	9034	9010	9034	9034	9030	560	904						5 3/8	5
141		HEAD FREE - R.A.	9027	90RA-T	9027		9027	9027			TC1013						5 25/32	5 1/8
142		INTERBOROUGH R. T.	9050	90RT	9050		9050	9050	902					77			5	5
143		LEHIGH VALLEY															5	5
144		N.Y.C. (DUDLEY)		90DY					901								5 1/2	5
145		UNION PACIFIC	9023		9023	9003	9023	9023			901						5 3/4	5 3/8
146	91	LACKAWANNA		91- DL	9133		9133	9133	911								5 1/4	5 3/8
147	92	FRICTIONLESS		304													5 7/16	5 3/8
148	93	FRICTIONLESS		93-NH-F					932								6 1/8	5 1/2
149	95	ASCE							950					267			5 9/16	5 9/16
150		BOSTON & ALBANY															5 1/32	5 1/2
151		W & H RY. (DUDLEY)		95-DY					951								5 1/32	5 1/2
152	97	FRICTIONLESS		97-CO-F													5 7/8	5 9/64
153	98	FRICTIONLESS		98-PS-F													5 27/32	5
154	100	ARA-A	10020	100RA	10020		10020	10020	10031	565	1003	10020					6	5 1/2
155		ARA-B	10030	100RB	10030		10030	10030	10032	564	1002	10030					5 41/64	5 9/64
156		AREA	10025	100RE	10025		10025	10025			10025	10025					6	5 3/8
157		ASCE	10040	100 AS	10040	10001	10040	10040	1000	536				247			5 3/4	5 3/4
158		CHI. & NOR. WESTERN	10035	100-OM	10035		10035	10035	1006			10035					5 45/64	5 9/64
159		ELGIN JOLIET & EAST.			10050		10050	10050									5 9/16	5
160		GREAT NORTHERN	10036	100GN	10036		10036	10036	1008								5 3/4	5
161		HEAD FREE - R.A.		100RA-T											10004	136	6 5/32	5 1/2
162		HEAD FREE - R.E.		100RE-T													6 1/16	5 3/8
163		INTERBOROUGH R. T.	10005	100RT	10005		10005	10005	1005								5 3/4	5 3/4
164		N.Y., N.H. & H.	10034	100NH	10034	10004	10034	10034	1002					100			6	5 1/2
165		NEW YORK CENTRAL		100-DY	10022	10003	10022	10022	1001								6	5 1/2
166		PENNSYLVANIA	10031	100PS	10031		10031	10031	10030	558		10031					5 11/16	5
167		PENNSYLVANIA	10033	100PR	10033	10002	10033	10033	1003	520				96			5 1/2	5 1/2
168		READING	10032	100RG	10032		10032	10032	1007								5 5/8	5 3/8
169		R.W. HUNT.															6	5 1/2
170		SANDBERG															5 3/4	6 1/4

Table of Rail Sections - AREA Committee 1B (Dimensions in Inches)

1	A	B	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF
2								Base			Gage	Top	Bottom	Head	Head	
3			Head	Web	Head	Web	Base	Edge	Head	Web	Corner	Fillet	Fillet	Side	Bottom	Base
4	Wt. / Yd.		Width	Thickness	Height	Height	Height	Thickness	Radius	Radius	Radius	Radius	Radius	Slope	Angle	Angle
5	lbs.	Type	HD	W	HH	WH	BH	T	RH	RW	R1	R2	R3	S	A1	A2
138		GREAT NORTHERN	2 5/8	9/16	1 15/32	2 7/8	1 1/32		12	14	7/16	7/16	5/8	1/16 to 1	13 deg.	13 deg.
139		GREAT NORTHERN	2 5/8	19/32	1 15/32	2 7/8	1 1/32							5 deg.	13 deg.	13 deg.
140		GREAT NORTHERN	2 5/8	5/8	1 1/2	2 7/8	1		14	14	5/8	3/8	3/8	5 deg.	13 deg.	13 deg.
141		HEAD FREE - R.A.	2 31/64	9/16	1 5/8	3 5/32	1		14	14	3/8	3/8		1/16 to 1	1 to 4; U = 54°	1 to 4
142		INTERBOROUGH R. T.	2 7/8	11/16	1 25/32	2 11/32	7/8	5/32	12	9	1/2	1/4	1/4	8 deg.	13 deg.	13 deg.
143		LEHIGH VALLEY	2 3/4	5/8	1 53/64	2 15/64	15/16	1/3	12	9	1/2	1/4	1/4	5 deg.	14 deg.	14 deg.
144		N.Y.C. (DUDLEY)	2 21/32	9/16	1 1/2	3 1/32	31/32		14	14	5/16	1/2	1	1/16 to 1	1 to 4	1 to 4
145		UNION PACIFIC	2 3/4	17/32	1 1/2	3 3/8	7/8							VERT.	13 deg.	13 deg.
146	91	LACKAWANNA	2 5/8	5/8	1 41/64	2 11/16	59/64		10	8	7/16	1/4	1/4	4 deg.	13 deg.	13 deg.
147	92	FRICTIONLESS	1 15/16	5/8	2 3/32	2 5/16	1 1/32							4 deg.	13 deg.	13 deg.
148	93	FRICTIONLESS	2 1/8	19/32	1 13/16	3 3/8	15/16							3/16 tol 1/4	13 deg.	13 deg.
149	95	ASCE	2 11/16	9/16	1 41/64	2 63/64	15/16		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
150		BOSTON & ALBANY	3	5/8	1 9/16	2 15/32	1	5/16	14	14	5/16	1/2	5/16	4 deg.	14 deg.	14 deg.
151		W & H RY. (DUDLEY)	3	5/8	1 9/16	2 15/32	1		14	14	5/16	1/2	5/16	1/16 to 1	1 to 4	1 to 4
152	97	FRICTIONLESS	2 1/4	9/16	1 15/16	2 55/64	1 5/64							1/4 to 1 1/4	13 deg.	13 deg.
153	98	FRICTIONLESS	2 1/2	9/16	1 31/32	2 25/32	1 3/32							15° 30'	15 deg.	13 deg.
154	100	ARA-A	2 3/4	9/16	1 9/16	3 3/8	1 1/16		14	14	3/8	3/8	3/8	1/16 to 1	1 to 4	1 to 4
155		ARA-B	2 21/32	9/16	1 45/64	2 55/64	1 5/64		12	12	3/8	5/16	5/16	3 deg.	13 deg.	13 deg.
156		AREA	2 11/16	9/16	1 21/32	3 9/32	1 1/16		14	14	3/8	3/8	5/8	1/16 to 1	1 to 4	1 to 4
157		ASCE	2 3/4	9/16	1 45/64	3 5/64	31/32		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
158		CHI. & NOR.WESTERN	2 9/16	9/16	1 39/64	2 61/64	1 9/64		12	12	3/8	5/16	5/16	3 deg.	13 deg.	13 deg.
159		ELGIN JOLIET & EAST.	2 21/32	9/16	1 37/64	2 51/64	1 3/16							1/16 to 1	1 to 4	1 to 4
160		GREAT NORTHERN	2 3/4	9/16	1 5/8	3	1 1/8							1/16 to 1	1 to 4	1 to 4
161		HEAD FREE - R.A.	2 11/16	9/16	1 23/32	3 3/8	1 1/16		14	14	3/8	3/8		1/16 to 1	1 to 4; U = 49°	1 to 4
162		HEAD FREE - R.E.	2 39/64	9/16	1 23/32	3 9/32	1 1/16							1/16 to 1	1 to 4; U = 57°	1 to 4
163		INTERBOROUGH R. T.	2 7/8	9/16	1 45/64	3 5/64	31/32		12	12	1/2	1/4	1/4	8 deg.	13 deg.	13 deg.
164		N.Y., N.H. & H.	2 3/4	19/32	1 23/32	3 11/32	15/16		12	12	7/16	1/4	1/4	4 deg.	13 deg.	13 deg.
165		NEW YORK CENTRAL	3	19/32	1 5/8	3 13/32	31/32		14	14	5/16	1/2	5/16	1/16 to 1	1 to 4	1 to 4
166		PENNSYLVANIA	2 43/64	9/16	1 13/16	2 25/32	1 3/32		10	10	7/16	5/16	5/16	3/32 to 1	15 deg.	13 deg.
167		PENNSYLVANIA	2 13/16	5/8	1 7/8	2 11/16	15/16	11/64	10	8	7/16	1/4	1/4	4 deg.	13 deg.	13 deg.
168		READING	2 21/32	9/16	1 45/64	2 55/64	1 1/16		12	12	7/16	5/16	5/16	3/64 to 1	13 deg.	13 deg.
169		R.W. HUNT.	2 9/16	9/16	1 19/32	3 21/64	1 5/64		12	12	3/8	3/8	3/8	VERT.	14 deg.	14 deg.
170		SANDBERG	3	17/32	1 7/8			3/8	5 3/4	VERT.	1/2			VERT.	20 deg.	20 deg.

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	AG	AH	AI	AJ	AK	AL	AM	AN	AO	AR
2			Head	Web	Neutral	Bolt						
3			Radius	Center	Axis	Center						
4	Wt. / Yd.		Width	Line	Distance	Line						
5	lbs.	Type	CH	WC	N	BC	REFERENCE					
138		GREAT NORTHERN		2 53/64	2.41		A, E, H, O					
139		GREAT NORTHERN				2 7/16	D					
140		GREAT NORTHERN		2 5/8	2.37	2 7/16	C, D, E, J					
141		HEAD FREE - R.A.		2 29/32			A, B, E, K					
142		INTERBOROUGH R. T.		2 7/32		2 1/16	A, D, E, I					
143		LEHIGH VALLEY				2 1/16	N					
144		N.Y.C. (DUDLEY)		3	2.52	2 3/8	A, C, D					
145		UNION PACIFIC		3		2 9/16	A, D, E					
146	91	LACKAWANNA		2 17/64	2.43	2 17/64	C, D, J					
147	92	FRICTIONLESS				2 3/16	D					
148	93	FRICTIONLESS		2 5/8		2 5/8	D, H					
149	95	ASCE		2 55/128	2.65	2 55/128	C, I					
150		BOSTON & ALBANY				2 9/32	N					
151		W & H RY. (DUDLEY)		2 19/32	2.36	2 9/32	C, H, I					
152	97	FRICTIONLESS		2 65/128		2 65/128	D, H					
153	98	FRICTIONLESS		2 31/64		2 31/64	D, H					
154	100	ARA-A		2 3/4	2.75	2 3/4	A, C, D, E, J					
155		ARA-B		2 65/128	2.63	2 65/128	A, C, D, E, J					
156		AREA		2 31/32	2.75	2 45/64	A, D, E, O					
157		ASCE		2 65/128	2.73	2 65/128	A, C, D, E, I, J					
158		CHI. & NOR.WESTERN		2 79/128	2.53	2 79/128	A, D, E, O					
159		ELGIN JOLIET & EAST.		2 75/128		2 75/128	D, H					
160		GREAT NORTHERN		2 13/16		2 5/8	A, D, E					
161		HEAD FREE - R.A.		2 15/16			A, B					
162		HEAD FREE - R.E.		2 31/32			A, B, E, K					
163		INTERBOROUGH R. T.		2 1/2	2.65	2 1/2	A, C, E, J, K					
164		N.Y., N.H. & H.		2 39/64	2.89	2 5/8	A, C, D, E, I, J					
165		NEW YORK CENTRAL		2 43/64	2.89	2 5/8	C, D					
166		PENNSYLVANIA		2 31/64	2.63	2 31/64	A, C, D, E, J, O					
167		PENNSYLVANIA		2 9/32		2 9/32	A, D, E, I					
168		READING		2 63/128	2.55	2 63/128	A, D, O					
169		R.W. HUNT.				2 95/128	N					
170		SANDBERO					N					

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R
2																		
3			Manufacturer and Section Number														Rail	Base
4	Wt. / Yd.					Old		Tennessee			Colorado			Maryl.			Height	Width
5	lbs.	Type	U.S. Steel	Bethlehem	Illinois	Illinois	Carnegie	TC & I	Lackawanna	Midvale	CF & I	Inland	Cambria	Penn.	Dominion	Algoma	H	B
171	101	LACKAWANNA	10133	101DL	10133		10133	10133	10130								5 7/16	5 3/8
172	105	LACKAWANNA	10533	105DL	10533		10533	10533	1052								6	5 3/8
173		DUDLEY	10524	105DY	10524		10524	10524				10524					6	5 1/2
174		NEW YORK CENTRAL		105-B	10522		10522	10522	1051								6	5 1/2
175	106	MISC.									1060						6 3/16	5 1/2
176	107	N.Y., N.H. & H.	10734	107NH	10734		10734	10734	1072								6 1/8	5 1/2
177	110	AREA	11025	110RE	11025		11025	11025			1100	11025					6 1/4	5 1/2
178		ASCE												268			6 1/8	6 1/8
179		C.T.A.	11050														7	6
180		GREAT NORTHERN	11036	110GN	11036		11036	11036				11036					6 1/2	5 1/2
181		HEAD FREE - AREA	11027	110RE-T	11027		11027	11027				11027					6 7/16	5 1/2
182		LEHIGH VALLEY	11033	110LV	11033		11033	11033									6	5 1/2
183	112	AREA	11228	112RE	11228		11228	11228			1121	11228					6 5/8	5 1/2
184		HEAD FREE - R.E.	11227	112RE-T	11227		11227	11227				11227HF					6 3/4	5 1/2
185		CB & Q - TR	11229		11229		11229	11229			1122						6 3/4	5 1/2
186	113	HEAD FREE - SO. PAC.	11327	113RE-T	11327		11327	11327			1130						6 13/16	5 1/2
187	115	AREA	11525	115RE	11525		11525	11525			1150	11525					6 5/8	5 1/2
188		D.R.G.W.									1155						6 5/8	5 1/2
189		DUDLEY	11522/11523	115DY	11523		11523	11523									6 1/2	5 1/2
190		MISCELL.															6	5 9/16
191	118	LACKAWANNA		118DL-M													6 1/2	5 3/8
192	119	AREA	11937								1190	11937					6 13/16	5 1/2
193	120	AREA			12025		12025	12025									6 1/2	5 3/4
194		MFG. STD.		120-MS													6 1/4	5 3/4
195		NEW YORK CENTRAL		120-DY					1201								7	6
196	122	CB (B&O)		122-CB													6 25/32	6
197	125	PENNSYLVANIA		308	12531		12531	12531	12530	584							6 1/2	5 1/2
198	125.5	FRICTIONLESS		125.5-PS-F													7	5 1/2
199	127	DUDLEY	12723	127DYM								12723					7	6 1/4
200		NEW YORK CENTRAL		127-DY	12722		12722	12722				12722					7	6 1/4
201	129	CB & Q - TR	12929		12929		12929	12929				12929					7 5/16	6
202	130	AREA	13025	130RE	13025		13025	13025			1300	13025					6 3/4	6

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF
2								Base			Gage	Top	Bottom	Head	Head	
3			Head	Web	Head	Web	Base	Edge	Head	Web	Corner	Fillet	Fillet	Side	Bottom	Base
4	Wt. / Yd.		Width	Thickness	Height	Height	Height	Thickness	Radius	Radius	Radius	Radius	Radius	Slope	Angle	Angle
5	lbs.	Type	HD	W	HH	WH	BH	T	RH	RW	R1	R2	R3	S	A1	A2
171	101	LACKAWANNA	2 3/4	5/8	1 23/32	2 11/16	1 1/32		10	8	7/16	1/4	1/4	4 deg.	13 deg.	13 deg.
172	105	LACKAWANNA	2 3/4	5/8	1 23/32	3 1/4	1 1/32		10	8	7/16	1/4	1/4	4 deg.	13 deg.	13 deg.
173		DUDLEY	3	5/8	1 5/8	3 13/32	31/32		14	14	5/16	1/2	3/4	1/16 to 1	1 to 4	1 to 4
174		NEW YORK CENTRAL	3	5/8	1 5/8	3 13/32	31/32		14	14	5/16	1/2	1	1/16 to 1	1 to 4	1 to 4
175	106	MISC.	2 21/32	19/32	1 3/4	3 3/8	1 1/16							1 to 40	1 to 4	1 to 4
176	107	N.Y., N.H. & H.	2 3/4	19/32	1 23/32	3 11/32	1 1/16		12	12	7/16	1/4	1/4	4 deg.	13 deg.	13 deg.
177	110	AREA	2 25/32	19/32	1 23/32	3 13/32	1 1/8		14	14	3/8	3/8	5/8	1/16 to 1	1 to 4	1 to 4
178		ASCE	2 7/8	37/64	1 25/32	3 11/32	1		12	12	5/16	1/4	1/4	VERT.	13 deg.	13 deg.
179		C.T.A.	2 3/4	9/16	1 7/8	4 5/16	13/16							3° 34' 30"	14 deg.	9 deg.
180		GREAT NORTHERN	2 3/4	19/32	1 5/8	3 3/4	1 1/8		14	14	3/8	1/2	5/8	1/16 to 1	1 to 4	1 to 4
181		HEAD FREE - AREA	2 11/16	19/32	1 29/32	3 13/32	1 1/8		14	14	3/8	3/8		1/16 to 1	to 4; U = 55° 30	1 to 4
182		LEHIGH VALLEY	2 7/8	19/32	1 7/8	3 1/16	1 1/16							4 deg.	1 to 4	1 to 4
183	112	AREA	2 23/32	19/32	1 11/16	3 13/16	1 1/8		24	10 & 23	1 & 1/4	3/8	5/8	1 to 40	1 to 4	1 to 4
184		HEAD FREE - R.E.	2 11/16	19/32	1 13/16	3 13/16	1 1/8		14	10 & 23	3/8	3/8		1 to 40	1 to 4; U = 58°	1 to 4
185		CB & Q - TR	2 1/2	5/8	1 3/4	3 7/8	1 1/8							1 to 40	to 4; U = 77° 45	1 to 4
186	113	HEAD FREE - SO. PAC.	2 11/16	19/32	1 7/8	3 13/16	1 1/8		14	10 & 23	1 & 3/8	3/8		1 to 40	1 to 4; U = 58°	1 to 4
187	115	AREA	2 23/32	5/8	1 11/16	3 13/16	1 1/8		10	3 & 14	1 1/2 & 3/8	3/4	3/4	1 to 40	1 to 4	1 to 4
188		D.R.G.W.	2 23/32	3/4	1 11/16	3 13/16	1 1/8							4 deg.	13 deg.	13 deg.
189		DUDLEY	3	5/8	1 11/16	3 3/4	1 1/16		14	14	5/16	1/2	3/4	1/16 to 1	1 to 4	1 to 4
190		MISCELL.	2 15/16	21/32	1 7/8	3 1/16	1 1/16							1/16 to 1	1 to 4	1 to 4
191	118	LACKAWANNA	2 7/8	5/8	1 29/32	3 1/2	1 3/32							3.25°	13 deg.	13 deg.
192	119	AREA	2 21/32	5/8	1 7/8	3 13/16	1 1/8		14	3 & 14	1 1/2 & 9/16	3/4	3/4	1 to 40	1 to 4	1 to 4
193	120	AREA	2 7/8	5/8	1 25/32	3 17/32	1 3/16							1/16 to 1	1 to 4	1 to 4
194		MFG. STD.	2 7/8	5/8	1 29/32	3 5/32	1 3/16	1/8	12	12	7/16	3/8	3/8	4 deg.	14 deg.	14 deg.
195		NEW YORK CENTRAL	3	21/32	1 5/8	4 5/16	1 1/16		14	20	5/16	1/2	1	1/16 to 1	1 to 4	1 to 4
196	122	CB (B&O)	2 15/16	21/32	1 15/16	3 39/64	1 15/64	1/8	10	3 & 14	1 1/4 & 3/8	3/4	3/4	1 to 20	1 to 2 3/4	1 to 2 3/4, 1 to 13.7
197	125	PENNSYLVANIA	3	21/32	1 7/8	3 13/32	1 7/32		12	16	7/16	1/2	3/4	VERT.	18 deg.	14 deg.
198	125.5	FRICTIONLESS	1 13/16	11/16	2 3/8	3 13/32	1 7/32							VERT.	18 deg.	14 deg.
199	127	DUDLEY	3	21/32	1 11/16	4 5/32	1 5/32							1/16 to 1	1 to 4	1 to 4
200		NEW YORK CENTRAL	3	21/32	1 11/16	4 5/32	1 5/32		14	18	5/16	1/2	3/4	1/16 to 1	1 to 4	1 to 4
201	129	CB & Q - TR	2 5/8	21/32	1 27/32	4 9/32	1 3/16							1 to 40	1 to 4	1 to 4
202	130	AREA	2 15/16	21/32	1 27/32	3 11/16	1 7/32		14	14	3/8	1/2	3/4	1/16 to 1	1 to 4	1 to 4

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Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	8	AG	AH	AI	AJ	AK	AL	AM	AN	AO	AR
2			Head	Web	Neutral	Bolt						
3			Radius	Center	Axis	Center						
4	Wt. / Yd.		Width	Line	Distance	Line						
5	lbs.	Type	CH	WC	N	BC	REFERENCE					
171	101	LACKAWANNA		2 3/8	2.46	2 3/8	A , C , E					
172	105	LACKAWANNA		2 21/32	2.79	2 21/32	A , C , E					
173		DUDLEY		3 1/8	2.88		A , E ,O					
174		NEW YORK CENTRAL		3 1/8	2.88	2 5/8	C , D					
175	106	MISC.		2 15/16			H					
176	107	N.Y., N.H. & H.		2 47/64	2.78	2 47/64	A , C , E					
177	110	AREA		3 1/8	2.83		A , E ,O					
178		ASCE		2 43/64			I					
179		C.T.A.					A					
180		GREAT NORTHERN		3 1/4	2.90		A , E ,O					
181		HEAD FREE - AREA		3 1/8			A , B , E , K					
182		LEHIGH VALLEY		2 15/16		2 3/4	A , D , E					
183	112	AREA		3 3/4			A , E , K					
184		HEAD FREE - R.E.		3 3/4			A , B , E					
185		CB & Q - TR		3 1/8			A , E , K					
186	113	HEAD FREE - SO. PAC.	1 19/32	3 3/4			A , B , E					
187	115	AREA	1 1/4	3 1/4	2.98		A , E , G					
188		D.R.G.W.		3 1/32			E					
189		DUDLEY		3 3/8	3.03		A , E ,O					
190		MISCELL.		2 15/16			H					
191	118	LACKAWANNA		3 13/32			E					
192	119	AREA	1 1/4	3 1/4	3.12		G					
193	120	AREA		3 1/4			H					
194		MFG. STD.		2 49/64		2 49/64	H , J					
195		NEW YORK CENTRAL		3 7/8	3.12	3	C , D , E					
196	122	CB (B&O)	1 15/64	3 21/64	3.18		K					
197	125	PENNSYLVANIA		3 1/4	3.00	2 59/64	C , D , E					
198	125.5	FRICTIONLESS		3 1/4		2 3/4	D , H					
199	127	DUDLEY		3 7/8			A , H					
200		NEW YORK CENTRAL		3 7/8	3.10		E ,O					
201	129	CB & Q - TR		3 1/2			A , E					
202	130	AREA		3 3/8	3.03		A , E ,O					

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R
2			Manufacturer and Section Number														Rail Height	Base Width
3																		
4	Wt. / Yd.					Old		Tennessee			Colorado			Maryl.				
5	lbs.	Type	U.S. Steel	Bethlehem	Illinois	Illinois	Carnegie	TC & I	Lackawanna	Midvale	CF & I	Inland	Cambria	Penn.	Dominion	Algoma	H	B
203		HEAD FREE - P.S.		130PS-T													6 13/16	5 1/2
204		HEAD FREE - R.E.	13027	130RE-T	13027		13027	13027				13027			13001	138	6 13/16	6
205		PHIL. & READING		130RG													6 27/32	6
206		PENNSYLVANIA	13031	130PS	13031		13031	13031	13030	589	1302	13031					6 5/8	5 1/2
207	131	AREA	13128	131RE	13128		13128	13128			1311	13128					7 1/8	6
208		HEAD FREE - R.E.	13127														7 1/4	6
209	132	AREA	13225	132RE	13225		13225	13225			1321	13225					7 1/8	6
210		HEAD FREE - SO. PAC.	13227	132RE-T	13227		13227	13227			1320						7 5/16	6
211	133	AREA	13331	133RE	13331		13331	13331			1330	13331					7 1/16	6
212	135	CENTRAL OF NJ		135CR													6 1/2	6
213	136	AREA	13637	136RE							1360						7 5/16	6
214		LEHIGH VALLEY	13633	136LV	13633		13633	13633									7	6 1/2
215		LEHIGH VALLEY		136-LV													7 3/8	6 1/2
216		LEHIGH VALLEY		136-LV-M													7	6 1/2
217		NEW YORK CENTRAL		136NYC													7 9/32	6 1/4
218	140	AREA/PS	14031	140RE	14031		14031	14031									7 5/16	6
219	152	PENNSYLVANIA	15222	152PS	15224		15224	15224									8	6 3/4
220	155	PENNSYLVANIA	15531	155PS	15531		15531	15531									8	6 3/4

Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

1	A	B	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF
2								Base			Gage	Top	Bottom	Head	Head	
3			Head	Web	Head	Web	Base	Edge	Head	Web	Corner	Fillet	Fillet	Side	Bottom	Base
4	Wt. / Yd.		Width	Thickness	Height	Height	Height	Thickness	Radius	Radius	Radius	Radius	Radius	Slope	Angle	Angle
5	lbs.	Type	HD	W	HH	WH	BH	T	RH	RW	R1	R2	R3	S	A1	A2
203		HEAD FREE - P.S.	3	21/32	2 3/16	3 3/8	1 7/32							STR.	18°; U = 58° 30'	14 deg.
204		HEAD FREE - R.E.	2 27/32	21/32	2 1/32	3 11/16	1 7/32		14	14	3/8	1/2		1/16 to 1	1 to 4; U = 61°	1 to 4
205		PHIL. & READING	2 15/16	21/32	1 15/16	3 11/16	1 7/32							1/16 to 1	1 to 4	1 to 4
206		PENNSYLVANIA	3	11/16	2	3 13/32	1 7/32		12	16	7/16	1/2	3/4	VERT.	18 deg.	14 deg.
207	131	AREA	3	21/32	1 3/4	4 3/16	1 3/16		24	10 & 23	1 & 1/4	1/2	3/4	1 to 40	1 to 4	1 to 4
208		HEAD FREE - R.E.	2 31/32	21/32	1 7/8	4 3/16	1 3/16	1/8	14	10 & 23	3/8	1/2		1 to 40	to 4; U = 60° 30	1 to 4
209	132	AREA	3	21/32	1 3/4	4 3/16	1 3/16	1/8	10	8 & 16	1 1/4 & 3/8	3/4 & 5/16	7/8	1 to 40	1 to 4	1 to 4
210		HEAD FREE - SO. PAC.	2 31/32	21/32	1 15/16	4 3/16	1 3/16	1/8	14	10 & 23	1 & 3/8	1/2		1 to 40	to 4; U = 60° 30	1 to 4
211	133	AREA	3	11/16	1 15/16	3 15/16	1 3/16	1/8	10	8 & 16	1 1/4 & 3/8	3/4 & 7/16	3/4	1 to 14.3	1 to 3	1 to 4.011
212	135	CENTRAL OF NJ	3 5/32	3/4	2	3 9/32	1 7/32							4 deg.	14 deg.	14 deg.
213	136	AREA	2 15/16	11/16	1 15/16	4 3/16	1 3/16	1/8	14	8 & 20	1 1/4 & 9/16	3/4 & 5/16	3/4	1 to 40	1 to 4	1 to 4
214		LEHIGH VALLEY	2 15/16	21/32	1 7/8	3 7/8	1 1/4							4 deg.	1 to 4	1 to 4
215		LEHIGH VALLEY	2 15/16	11/16	1 25/32	4 3/8	1 7/32							4 deg.	1 to 4	1 to 4
216		LEHIGH VALLEY	2 15/16	11/16	1 7/8	3 7/8	1 1/4							4 deg.	1 to 4	1 to 4
217		NEW YORK CENTRAL	2 15/16	11/16	1 7/8	4 5/32										
218	140	AREA/PS	3	3/4	2 1/16	4 1/16	1 3/16	1/8	10	8 & 27	1 1/4 & 3/8	3/4 & 7/16	3/4	4 deg.	1 to 3	1 to 4
219	152	PENNSYLVANIA	3	11/16	1 27/32	4 7/8	1 9/32		24	6&30	1 & 1/4	1/2	3/4	1 to 40	14 deg.	14 deg.
220	155	PENNSYLVANIA	3	3/4	2 1/16	4 21/32	1 9/32							4 deg.	18° 26' 10"	14 deg.

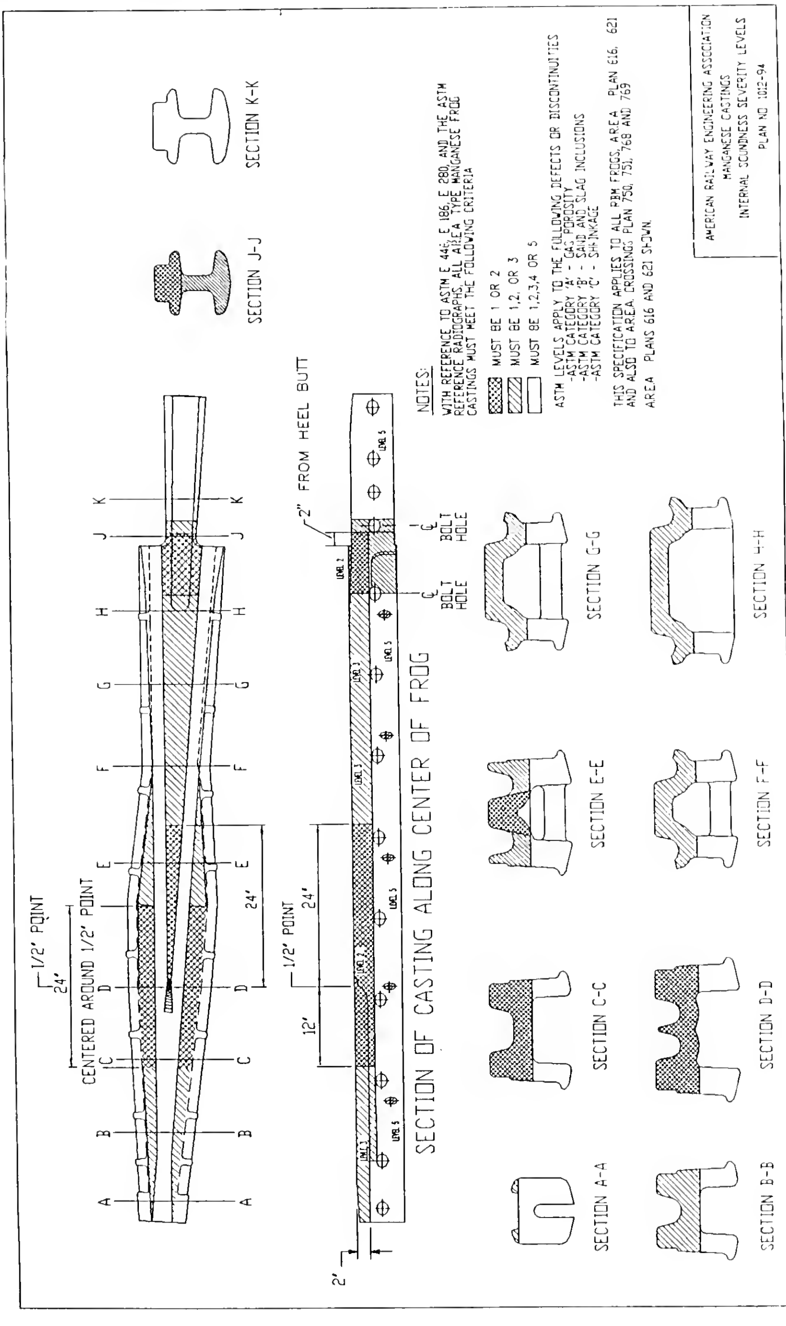
Table of Rail Sections - AREA Committee 18 (Dimensions in Inches)

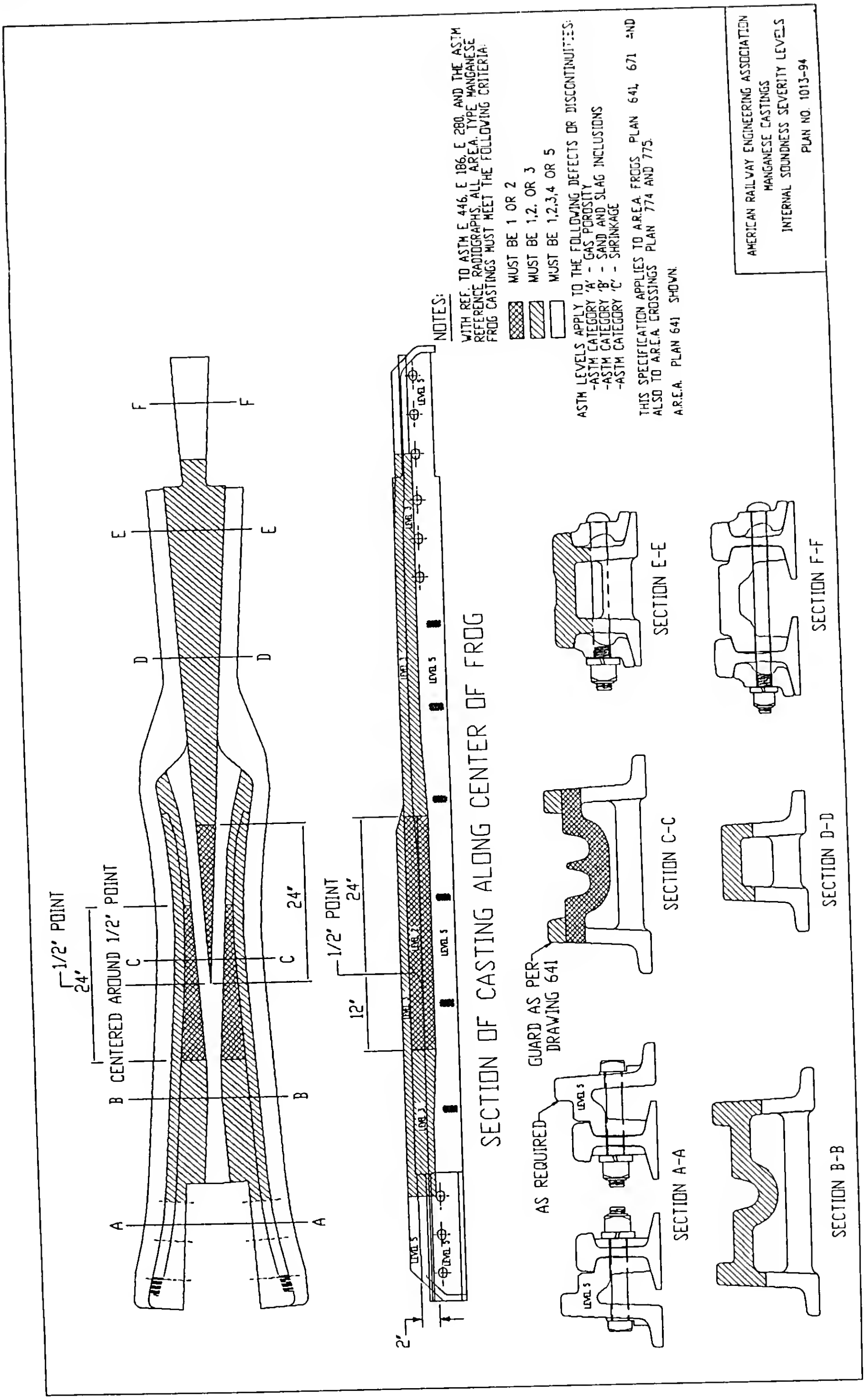
1	A	B	AG	AH	AI	AJ	AK	AL	AM	AN	AO	AR
2			Head	Web	Neutral	Bolt						
3			Radius	Center	Axis	Center						
4	Wt. / Yd.		Width	Line	Distance	Line						
5	lbs.	Type	CH	WC	N	BC	REFERENCE					
203		HEAD FREE - P.S.		3 1/4			A					
204		HEAD FREE - R.E.		3 3/8	3.08		A , B , E ,O					
205		PHIL. & READING		3 3/8			E , K					
206		PENNSYLVANIA		3 1/4	3.09	2 3/4	A , D , E ,O					
207	131	AREA		4 1/4			A , E					
208		HEAD FREE - R.E.		4 1/4			A , B , K					
209	132	AREA	1 1/2	3 7/8	3.20		A , E , G					
210		HEAD FREE - SO. PAC.	1 13/16	4 1/4			A , B , E					
211	133	AREA	1.4	3 3/4	3.20		A , E , G					
212	135	CENTRAL OF NJ		3		2 55/64	D , H					
213	136	AREA	1.4	3 7/8	3.35		G					
214		LEHIGH VALLEY		3 1/2		3 1/16	A , D , E					
215		LEHIGH VALLEY		3 7/8			E					
216		LEHIGH VALLEY		3 1/2			E , H					
217		NEW YORK CENTRAL					K					
218	140	AREA/PS	1.4	4	3.37		A , E , G					
219	152	PENNSYLVANIA	1 13/16	5	3.50		A , E ,0					
220	155	PENNSYLVANIA		4 11/16			A , E					

KEY FOR REFERENCES:

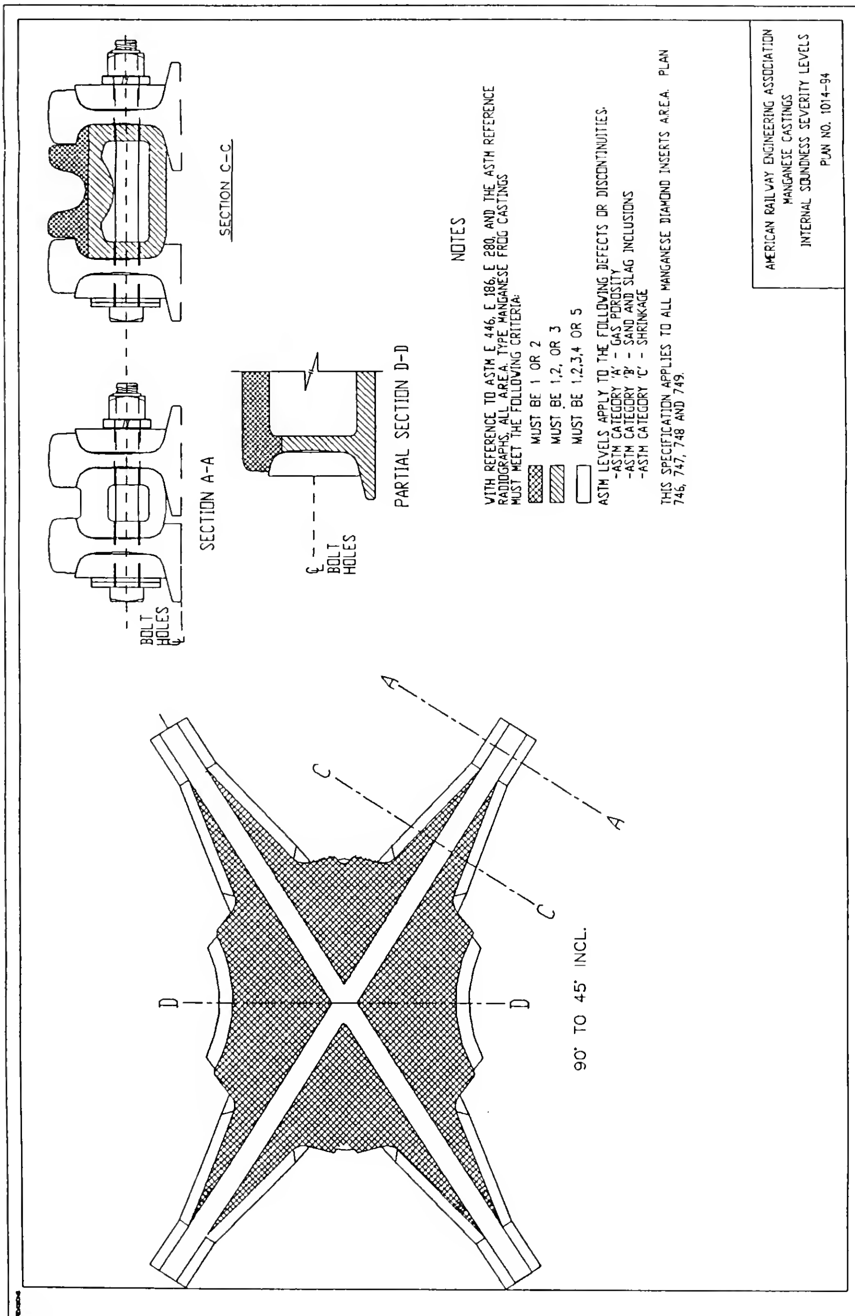
A	AREA PLAN 1001 - 55
B	AREA PLAN 1004 - 52
C	Rails & Fastenings 1916, Lackawanna Steel Co.
D	Handbook of Railroad Track Standards (Army)
E	1948 Railway Engineering & Maintenance Cyclopedia
F	D. Wemmer - Santina & Thompson
G	AREA VOL. 1
H	Table from Du-Wel Steel
I	Rails & Accessories - The PA Stl. Co., MD Stl. Co.
J	Rails 1914 Carnegie Steel Company
K	Correspondence From Bethlehem Steel Corp.
L	1955 Railway Engineering & Maintenance Cyclopedia
N	Railway Track and Track Work, E.E. Russel Tratman, 1897 and 1909 editions
O	Illinois Pocket Companion, Illinois Steel, 1934

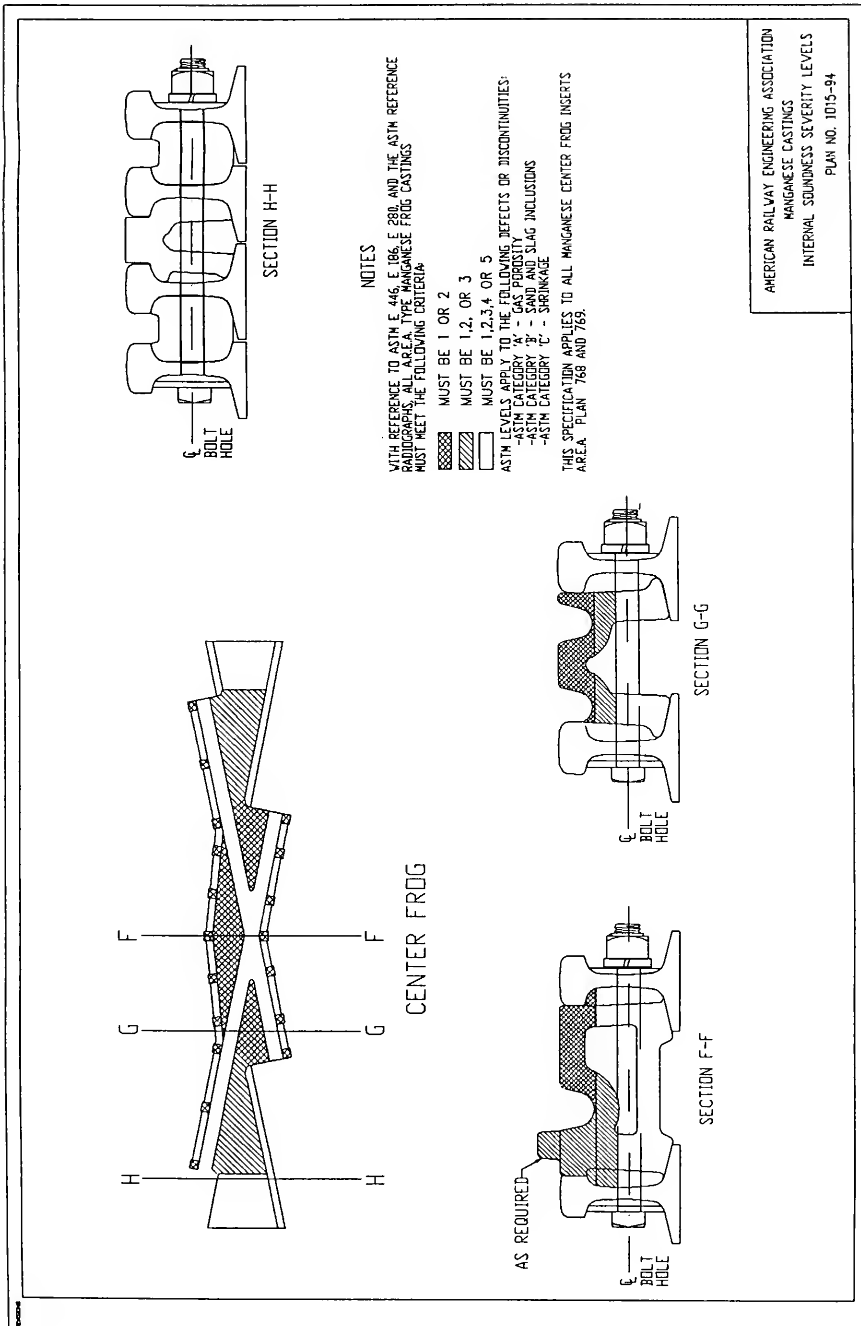
Radiographic Testing of Manganese Frog Castings—Internal Soundness Severity Levels for Various Castings. Replace Plan No's. 1012-92 through 1017-92 (6 drawings) with the following:

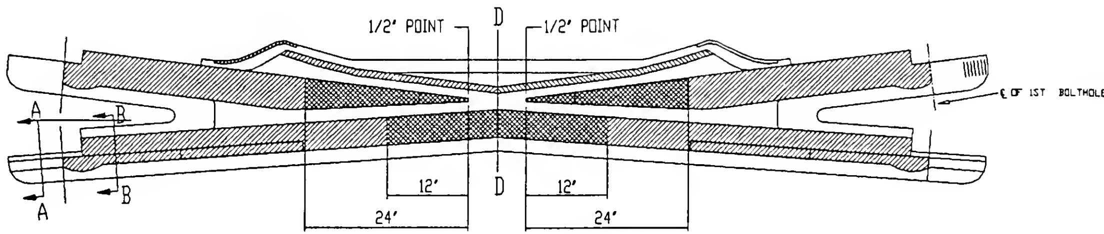




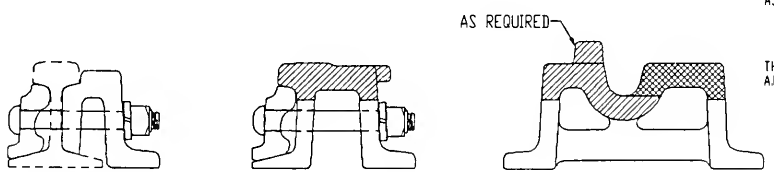
NOTES:
WITH REF. TO ASTM E 446, E 186, E 280, AND THE ASTM REFERENCE RADIOGRAPHS, ALL A.R.E.A. TYPE MANGANESE FROG CASTINGS MUST MEET THE FOLLOWING CRITERIA.
MUST BE 1 OR 2
MUST BE 1,2, OR 3
MUST BE 1,2,3,4 OR 5
ASTM LEVELS APPLY TO THE FOLLOWING DEFECTS OR DISCONTINUITIES:
-ASTM CATEGORY 'A' - GAS POROSITY
-ASTM CATEGORY 'B' - SAND AND SLAG INCLUSIONS
-ASTM CATEGORY 'C' - SHRINKAGE
THIS SPECIFICATION APPLIES TO A.R.E.A. FROGS PLAN 641, 671 AND ALSO TO A.R.E.A. CROSSINGS PLAN 774 AND 775.
A.R.E.A. PLAN 641 SHOWN.
AMERICAN RAILWAY ENGINEERING ASSOCIATION
MANGANESE CASTINGS
INTERNAL SOUNDNESS SEVERITY LEVELS
PLAN NO. 1013-94
1/2" POINT
24"
CENTERED AROUND 1/2" POINT
24"
12"
2"
SECTION OF CASTING ALONG CENTER OF FROG
LEVEL 5
LEVEL 3
LEVEL 1
GUARD AS PER DRAWING 641
AS REQUIRED
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F


SECTION C-C
SECTION A-A
BOLT HOLES ℄
PARTIAL SECTION D-D
℄ BOLT HOLES
NOTES
WITH REFERENCE TO ASTM E 446, E 186, E 280, AND THE ASTM REFERENCE RADIOGRAPHS, ALL A.R.E.A. TYPE MANGANESE FROG CASTINGS MUST MEET THE FOLLOWING CRITERIA:
MUST BE 1 OR 2
MUST BE 1,2, OR 3
MUST BE 1,2,3,4 OR 5
ASTM LEVELS APPLY TO THE FOLLOWING DEFECTS OR DISCONTINUITIES:
-ASTM CATEGORY 'A' - GAS POROSITY
-ASTM CATEGORY 'B' - SAND AND SLAG INCLUSIONS
-ASTM CATEGORY 'C' - SHRINKAGE
THIS SPECIFICATION APPLIES TO ALL MANGANESE DIAMOND INSERTS A.R.E.A. PLAN 746, 747, 748 AND 749.
AMERICAN RAILWAY ENGINEERING ASSOCIATION
MANGANESE CASTINGS
INTERNAL SOUNDNESS SEVERITY LEVELS
PLAN NO. 1014-94
A
A
C
C
D
D
90° TO 45° INCL.


SECTION H-H
℄ BOLT HOLE
NOTES
WITH REFERENCE TO ASTM E 446, E 186, E 280, AND THE ASTM REFERENCE RADIOGRAPHS, ALL A.R.E.A. TYPE MANGANESE FROG CASTINGS MUST MEET THE FOLLOWING CRITERIA:
MUST BE 1 OR 2
MUST BE 1,2, OR 3
MUST BE 1,2,3,4 OR 5
ASTM LEVELS APPLY TO THE FOLLOWING DEFECTS OR DISCONTINUITIES:
-ASTM CATEGORY 'A' - GAS POROSITY
-ASTM CATEGORY 'B' - SAND AND SLAG INCLUSIONS
-ASTM CATEGORY 'C' - SHRINKAGE
THIS SPECIFICATION APPLIES TO ALL MANGANESE CENTER FROG INSERTS A.R.E.A. PLAN 768 AND 769.
AMERICAN RAILWAY ENGINEERING ASSOCIATION
MANGANESE CASTINGS
INTERNAL SOUNDNESS SEVERITY LEVELS
PLAN NO. 1015-94
H
G
F
CENTER FROG
SECTION G-G
℄ BOLT HOLE
AS REQUIRED
SECTION F-F
℄ BOLT HOLE



CENTER FROG



SECTION A-A

SECTION B-B

SECTION D-D

NOTES

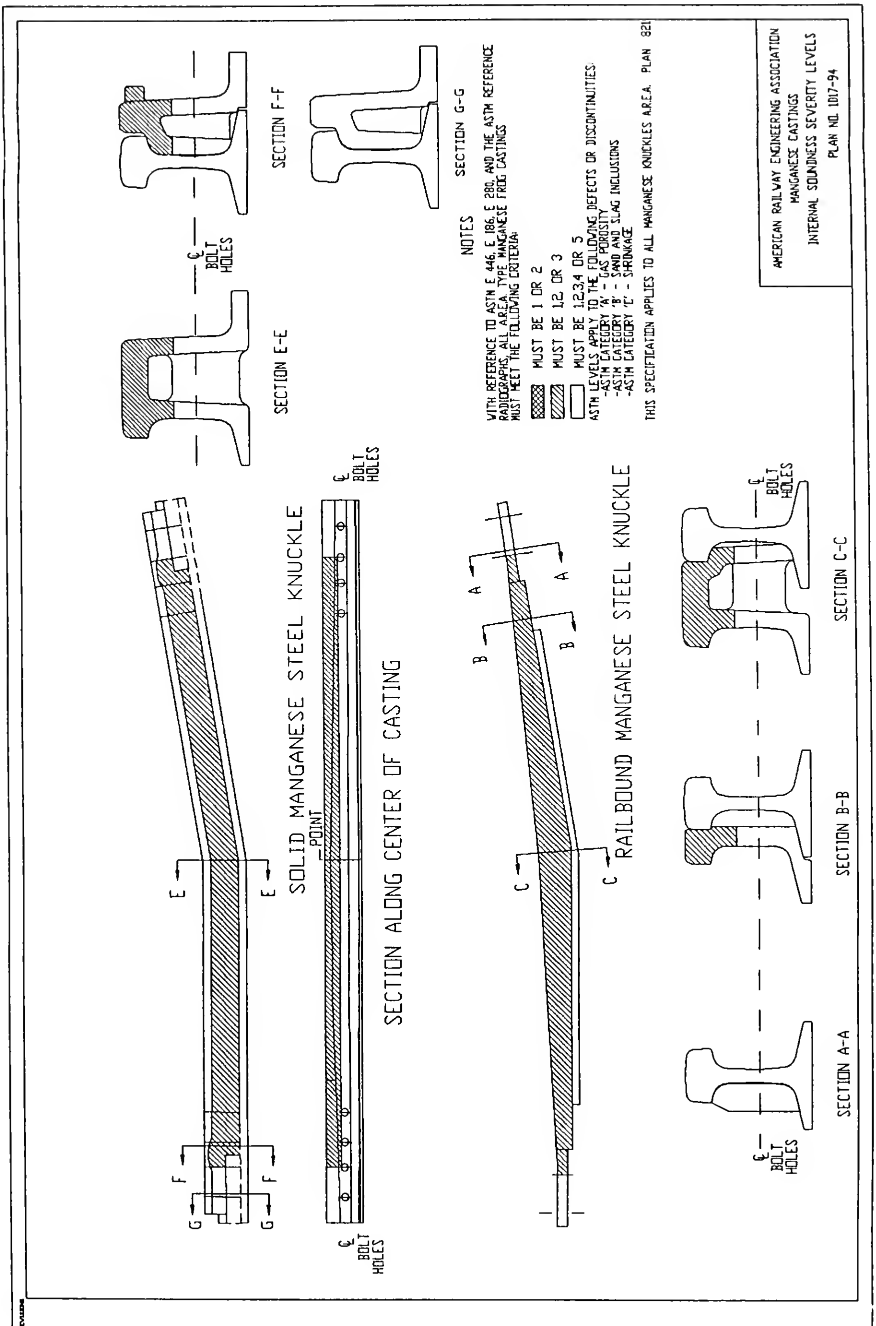
WITH REFERENCE TO ASTM E 446, E 186, E 280, AND THE ASTM REFERENCE RADIOGRAPHS, ALL A.R.E.A. TYPE MANGANESE FROG CASTINGS MUST MEET THE FOLLOWING CRITERIA:

- MUST BE 1 OR 2
- MUST BE 1,2, OR 3
- MUST BE 1,2,3,4 OR 5

ASTM LEVELS APPLY TO THE FOLLOWING DEFECTS OR DISCONTINUITIES:
- ASTM CATEGORY 'A' - GAS POROSITY
- ASTM CATEGORY 'B' - SAND AND SLAG INCLUSIONS
- ASTM CATEGORY 'C' - SHRINKAGE

THIS SPECIFICATION APPLIES TO ALL SOLID MANGANESE CENTER FROG INSERTS A.R.E.A. PLAN 774 AND 775.

AMERICAN RAILWAY ENGINEERING ASSOCIATION
MANGANESE CASTINGS
INTERNAL SOUNDNESS SEVERITY LEVELS
PLAN NO. 1016-94


SOLID MANGANESE STEEL KNUCKLE
POINT
SECTION ALONG CENTER OF CASTING
℄ BOLT HOLES
RAILBOUND MANGANESE STEEL KNUCKLE
SECTION A-A
SECTION B-B
SECTION C-C
SECTION E-E
SECTION F-F
SECTION G-G
NOTES
WITH REFERENCE TO ASTM E 446, E 186, E 280, AND THE ASTM REFERENCE RADIOGRAPHS, ALL A.R.E.A. TYPE MANGANESE FROG CASTINGS MUST MEET THE FOLLOWING CRITERIA:
MUST BE 1 OR 2
MUST BE 1,2, OR 3
MUST BE 1,2,3,4 OR 5
ASTM LEVELS APPLY TO THE FOLLOWING DEFECTS OR DISCONTINUITIES:
-ASTM CATEGORY 'A' - GAS POROSITY
-ASTM CATEGORY 'B' - SAND AND SLAG INCLUSIONS
-ASTM CATEGORY 'C' - SHRINKAGE
THIS SPECIFICATION APPLIES TO ALL MANGANESE KNUCKLES A.R.E.A. PLAN 821
AMERICAN RAILWAY ENGINEERING ASSOCIATION
MANGANESE CASTINGS
INTERNAL SOUNDNESS SEVERITY LEVELS
PLAN NO. 1017-94

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American Railway Engineering Association

May 1996

Volume 97, Bulletin 756

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American Railway Engineering Association

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D. E. Staplin, Editor

Contents

Front Cover: NDM Passenger Train enroute to Veracruz between Los Reyes and Ciudad Mendoza, Mexico

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THE RESTRUCTURING OF MEXICO'S RAILROADS

By: Lorenzo Reyes Retana*

The Mexican railroad system has played a distinct and important role throughout the country's independent history.

From its beginnings and up until the mid-20th century, the railroad system was a key factor in the integration of the country's political, economical, and social consolidation.

After that time, the rails helped the country to supply provisions, to industrialize, and to maintain price stability, thanks in part, to subsidized tariffs.

Nevertheless, since 1984 the railroads in Mexico started to lose a significant portion of their market share to other means of transportation. This was due, partly, to the ending of the subsidies mentioned before. The smaller revenues thus resulting diminished the availability of funds necessary to invest in the modernizing and upgrading of the railway's infrastructure, its telecommunications, and its rolling equipment.

Today, one of the main strategic objectives of the government in Mexico is to modernize its transportation system, in order to support economic competitiveness, within the globalization process. That is why it has been decided to undertake the restructuring of the railroad system.

In order to have a fairly clear picture of the restructuring process, let us take a look at the historical perspective of the railroads' development in Mexico.

Background

The history of Mexican railroads goes back to just a few years after England put its first passenger train into service, in 1830, between Manchester and Liverpool.

In 1837, at the beginning of the country's political independent life, the Mexican state gave private national interests the first concession to build a line that would link the Port of Veracruz to Mexico City. This line was completed in 1872.

A short time after its inauguration, the line was acquired by the English consortium, The Mexican Railroad Company, Ltd., with headquarters in London.

Later on, once the country had been politically organized as a representative, democratic federal republic, several national and foreign groups expressed their interest in building new lines and presented projects for different routes throughout the country.

At first, concessions were granted only to state governments and to private Mexican investors. This policy was short lived, since in 1880, two concessions were granted to investors from the United States.

During the period from 1874 to 1884, 3,300 miles of rail were built through concessions. The most important was the completion of the Mexico-Ciudad Juarez line.

Towards the end of the nineteenth century, there were a great many requests for concessions to build railroad lines in Mexican territory. At that time, there was no adequate policy to guide the design of routes, nor were there norms to regulate the width of the rails. As a result, in 1899, the First

*Director, Southeast Railway, Mexican Railway System

General Railroad Law went into effect. Two important issues were resolved by this document. The first issue referred to how railroad routes were to be built and operated and the second indicated which lines should be built to complete the railroad network.

And so, between 1885 and 1904, nearly 6,650 miles of rails were built, which when added to the lines already constructed, made a total of 10,332 rail miles.

By the end of 1910, a total of close to 12,000 miles of rail had been built, of which approximately 77% were wide rail lines and the rest were narrow rail lines.

By then, the railroads had not only surpassed the only overland alternative transportation mode (a network of roads for draft-pulled vehicles) but it had also significantly supported the commercial, social, and political growth of the country.

However, the dynamics of railroad transportation were halted by the military engagements during the decade of the Mexican Revolution between 1910–1920, in which time the railroad network was extensively employed. As a result, Mexico's railroads suffered severe damage to both its rails and bridges, and to the moving equipment. It became necessary to rebuild its infrastructure and to purchase new passenger and freight cars as well as locomotives.

In the light of this situation, during the 40-year period between 1911–1950, only 2,645 miles were added to existing rails, making a total of 14,582 miles. Additionally, the widening of narrow rails was undertaken.

With the advent of diesel engines at the end of the 1930's, Mexico started the replacement of its fleet of steam locomotives. That made it necessary, in turn, to finish the widening of the rails on those lines that still had narrow gauge, to reinforce embankments and bridges, and to replace the ties and ballast of its principal routes. It was also necessary to build supply depots as well as maintenance and repair workshops.

Finally, 2,000 miles of rail, mainly realignments and new links between existing lines, were built from 1951 to the present.

A great part of Mexico's rail network was built through concessions to private companies. These private companies were responsible for the rapid growth and expansion of the rail lines and became important railroad companies by buying, selling, leasing, or transferring of concessions. The National Railways of Mexico, known now as FNM, was formed in 1908, and 51% of its stock was held by the Federal Government. In 1948, the Organic Law, issued for the National Railways of Mexico, gave its legal status as a decentralized company of the Federal Government, creating the public non-profit organization that still exists to this day.

Current Situation

Mexico's railroad network is made up of 16,313 miles of rail, of which 12,625 miles are main lines, 2,750 are yards and sidings, and, 937 miles are private lines (Figure 1).

Forty percent of the principal rail lines meet modern specifications, that is, they are made of 100 to 136 lb./yd. caliber high quality continuously welded rails, supported on concrete ties. Another 35% of the railroad network has traditional specifications, with rails of 100 lb./yd. caliber or more, laid on wood ties.

Finally, 25% of the network are branch lines equipped with low caliber rails jointed and nailed on wood ties.

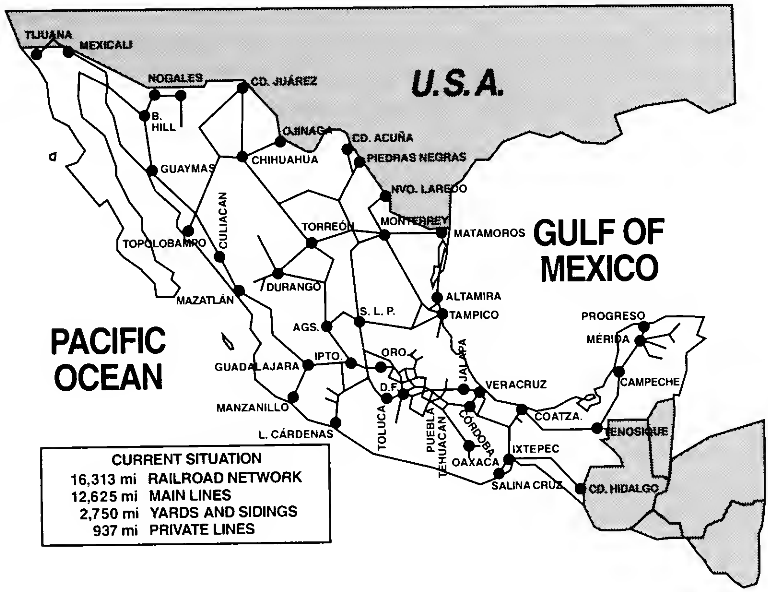


Figure 1.

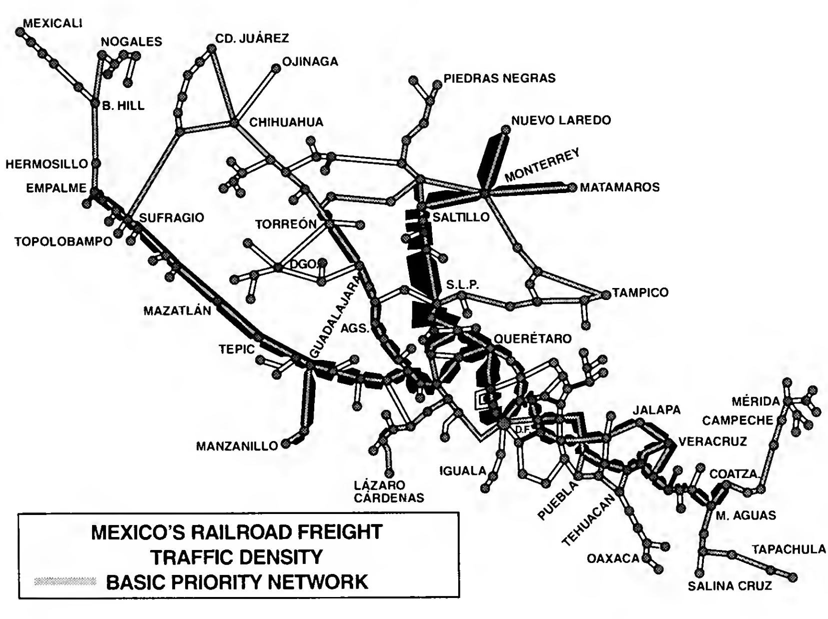


Figure 2.

Based on traffic volume, 7,000 miles of rail make up the Basic Priority Network, where the volume of freight, that produces approximately 85% of income, is currently moved (Figure 2).

In general, the norms and specifications of the line's components and of FNM's installations match those established by the American Railway Engineering Association (AREA).

With regard to the telecommunications infrastructure, close to 9,000 miles of railroad are covered by a microwave system. There are 17 ground satellite stations linking those areas not covered by the microwave telecommunications system, and there are thirty-eight fiber optic links that are in the process of being incorporated from the network of the national telephone long distance company.

Nearly 2,000 miles of track are equipped with CTC signaling systems.

This railroad network links most of the country's main cities in all but two states of the republic; included in this are the 10 most important seaports of the country, as well as 10 points along the border with the United States where freight is interchanged with three major American railroads.

The rolling stock consists of 1,261 locomotives and 27,000 operating freight cars.

The mining, steel, cement, sugar, fertilizer, petrochemical, paper, automobile, and grain transportation industries depend on the railroad, which has an active labor force of about 46,000 workers. This infrastructure is adequate to constitute the backbone of ground transportation in Mexico.

The total amount of freight transported in 1995 was a little over 50 million metric tons. This represents 20% of all merchandise transported by land in the country. However, this is not enough when compared to participation of other countries' railroad systems in their own land transportation markets. For example, the American railroads handle 37% of their country's land freight, and the Canadian railway system carries an even larger portion of its own ground transportation market.

The share of the market covered by the railway system in Mexico used to be larger. It started to decrease in 1985 as a result of the liberalization process of the economy and the shortage of investments needed for the adequate maintenance and upgrading of the railroad's infrastructure and equipment.

Therefore, in order to support the country's commercial and industrial competitiveness, in the globalization era in which we live, it is necessary to restructure the railway system in Mexico.

The main strategy of the process consists of promoting private sectors' participation, national and foreign, similar to the kind of participation that allowed the original, very forceful development of the railway infrastructure in Mexico.

New investments are expected to render profitable returns, and at the same time provide for the modernization of the railroads through the participation of experienced operators. Competition will induce a more market-oriented development and better overall services, in which safety and efficiency will be the rule.

The Restructuring Program

Prior to February 1995, the role of the private sector had been restricted by law. The restructuring process began, with the present President's initiative, to amend the legal framework in order to allow private capital participation in the operation of railroads.

In March of 1995, Congress passed an amendment to Constitutional Article 28, Paragraph Four, which formerly established the States' exclusiveness in the management of the railroads. The following May, Congress authorized the Regulating Law for the Railroad Service. This law establishes the basic legal framework for the private sector to participate directly in the operations of railroads.

Principally, this law establishes that the property of the right of way will remain with the Mexican government. The participation of the private sector will consist of concessions to be granted through public bidding for periods of up to 50 years, with the possibility of an extension for another 50 years.

Under this new law, foreign investors may hold more than 49% of stock with authorization by the National Commission for Foreign Investment.

The privatization process is steered and promoted by a committee chaired by the Secretary of Transportation and is composed of officers from the Ministry of Communications and Transportation, and the FNM.

This committee has selected two banks as financial agents, one American and one Mexican. These banks will help in the privatization effort by supervising the asset appraisals, integrating the data room, assessing the concessions, and heading the due diligence process.

The general guidelines for the bidding process were published recently. These guidelines provide detailed information on the following general concepts:

- New railroad companies will be formed and concessions will be granted for their operation. Rolling stock and locomotives will be properly distributed and fair values will be set by independent recognized consultants.
- There will be separate bidding for each of the new companies, and the successful bidder will be selected on the basis of fair pricing and the amount of investment proposed.

The process of privatization is well advanced and on schedule. In order to determine the best way to accomplish this process, extensive studies were undertaken on all past experiences around the world. Results concluded that a "tailor made" regional segmentation, similar to the American system, but responding to our country's particular needs, was the most adequate concept.

In order to insure a successful privatization, the companies will be transferred without any labor, financial, or environmental liabilities.

The FNM has already divided the existing railway system into three major profit centers, according to region. There are also plans to create an independent terminal company in Mexico City. The three railway systems are operating with a reasonable degree of independence and the planned independent terminal company can be created in due time.

The Mexican Railway System formed thus far, consists of the following (Figure 3):

The Northeast Railway, which has 2,500 miles of track, has 9 billion ton-miles of traffic reported in 1995, of which more than half is international. This railroad connects Mexico City with the Laredo Gateway as well as with the important seaports of Veracruz and Tampico on the Gulf of Mexico, and Lázaro Cárdenas on the Pacific Ocean.

The Pacific-North Rail, with 3,900 miles of track, links the metropolitan area of Mexico City with Guadalajara and the United States border through the cities of Mexicali, Nogales, Ciudad Juárez, and Piedras Negras. It also has lines to the ports of Manzanillo on the Pacific Ocean and Tampico on the Gulf of Mexico.

This railroad is the largest in length and traffic volume, but has a lower density in its routes than the Northeast Railway. Furthermore, 62% of its 10 billion ton-miles of traffic in 1995 was domestic.

The Southeast Railway, while only 1,400 miles long, and with 2 billion ton-miles of traffic, extends through a part of the country that shows great potential for growth. It connects Mexico City to the port of Veracruz. It also connects Mexico City and Veracruz to the ports of Coatzacoalcos and Salina Cruz through the Isthmus of Tehuantepec, and to the Yucatan peninsula.

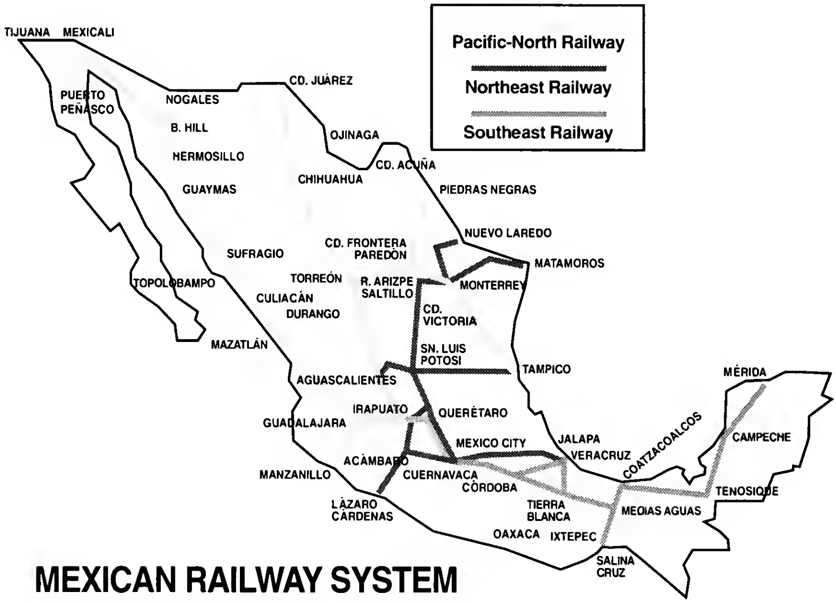


Figure 3.

Taking into account that these three regional railroads connect in Mexico City and that this terminal has the largest volume of traffic in the country, an independent terminal company is to be created to carry out switching and yard activities for all three regional railroads.

Moreover, regional segmentation anticipates the use of short lines for specific purposes. This is the case of the Chihuahua-Pacific Railroad which has interesting opportunities for tourism through the famous Copper Canyon (Figure 4).

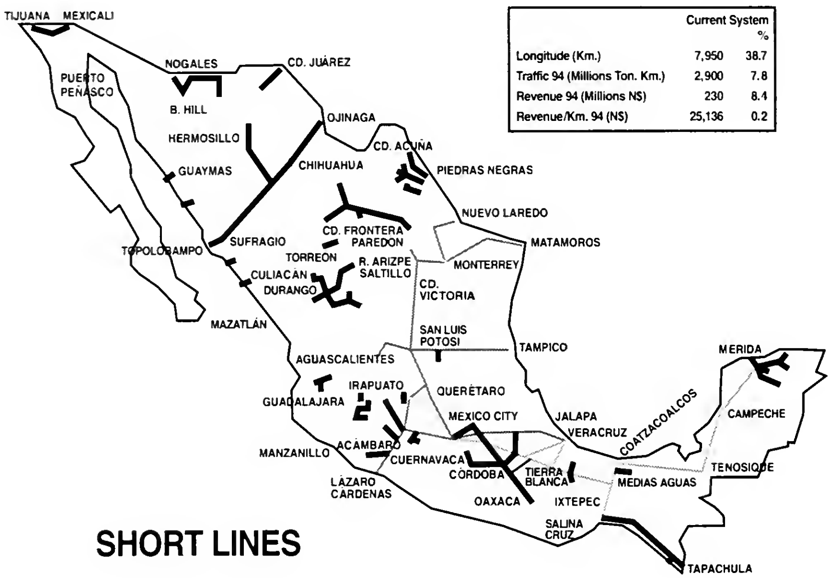
The rest of the short lines are stretches of the railroad network that can operate independently and, because of their traffic volume or their specialization, they only offer a profit potential to smaller companies.

Some of these lines can be concessioned to the regional railroads. Bidding will be made separately, in any case.

As regards to the labor issue, there is a great understanding of the privatization process on the part of the workers and their exclusive Railway Workers Union. There is also an attitude of cooperation and mutual respect between the union and the administration.

Insofar as the financial liabilities of FNM, as of 1995, the Federal Government has already absorbed them. The new companies will have clean balance sheets at the time of transfer to their private owners.

Environmental liabilities resulting from previous operations will remain the governments' responsibility and the new owners will be required by law to prevent any future damage to the environment. To this end, 50 environmental audits will have been completed by the time of the transferal and the corrective measures will already be under way.



	Current System	%
Longitude (Km.)	7,950	38.7
Traffic 94 (Millions Ton. Km.)	2,900	7.8
Revenue 94 (Millions N$)	230	8.4
Revenue/Km. 94 (N$)	25,136	0.2

Figure 4.

With this presentation, I hope I have been able to convey these particularly important aspects:

- Mexican railroads need to be modernized,
- The involvement of the private sector, which was essential for the establishment and growth of railroads in Mexico, is an adequate strategy,
- The privatization process is based on solid terms and is being conducted transparently, on schedule, and with no unforeseen obstacles that we cannot overcome and,
- Finally, the Mexican government has the political will and the determination to accomplish this process.

PRESIDENT'S ADDRESS

By: B. G. Willbrant*

Good morning again, and welcome to the 1996 AREA March Technical Conference. We have an exciting three days planned for you and I'm hoping you'll all enjoy the conference and the presentations. I want to thank the presenters for their time and effort.

It has certainly been a changing year in the industry as well as the AREA. I would first like to talk a little bit about the industry, but primarily my address this morning is going to be on the operation of the AREA within this past year. On the industry side, we started the year off with the merger of the Chicago Northwestern-Union Pacific, the merger of the Burlington Northern-Santa Fe, and the present proposed merger of the Union Pacific and the Southern Pacific with many other railroads involved in that merger. At this point in time, it's not been resolved and who knows where it will end up. Also, there has been early retirement buyouts on some of the railroads due to the mergers, as well as other roads, such as the NS, that recently went through an early retirement buyout. Conrail has one on the street right now. Of course, what this means is reductions in engineering staffs, a smaller group of engineers out there for this Association to draw from, and we'll discuss that a little bit later.

The industry growth, in general, this past year was nowhere near like 1994 where we had a boom year. Last March, when I got up here, myself and Mike Franke talked about the excellent results in the year 1994, how we were pushed to capacity. That was not the case in all parts of the country this past year. As a matter of fact, in our segment, things are down from the year before. In other areas, they were up but nowhere like in 1994.

It was certainly a changing year in the AREA. When I stood up here a year ago, we were in the process of interviewing for a permanent Executive Director. That mission was accomplished. We hired Dave Staplin, who had been Chief Engineer, Assistant Vice President for Amtrak. He's been in the industry for 24 years. He graduated from the University of Michigan with a BSCE. He has an MBA from Jacksonville University. He's been active in AAR committees, ASCE, Past President of Roadmasters and, of course, very active in AREA. We could not have found a better candidate to lead us into the future than Dave Staplin.

Dave Staplin came on board June 1 and worked on an interim basis for the month of June with John Robinson, who had been handling the duties since January 1 and John has been used on an as needed basis since that point in time when Dave got overloaded. Dave had very little time to get his feet on the ground because we had the Fall Technical Conference right around the corner. It was much earlier this year due to the conference being held in Anchorage, Alaska and the weather there. It was early September versus early October, so Dave had to get moving quite rapidly. And, of course, we had to set up the chairman's meeting as well as the Fall Technical Conference, as well as the train trip. Also it was felt that we needed a board meeting while we were there to keep the board informed and reach consensus on issues that needed to be addressed and resolved.

The Committee Chairmen's meeting was a little bit different this year. We broke it up into four break out sessions. One was to discuss a mission statement, which is just about finished. You'll be seeing it in the very near future. Another one was to discuss membership, what constitutes a member and I'll talk a little bit more about that later. Another was to speed up manual changes, and it is still continuing. In this day and age, we just feel that it takes too long to address manual changes. We must be much more proactive and move much quicker than we have been in the past. And the fourth was the Technical Conference with different formats and a call for paper process, which hopefully we'll have in place for next year's Technical Conference.

*Chief Engineer, Conrail

The fall conference was very successful. We had the Committee Chairmen's meeting. We had a board meeting that was to discuss the items and changes required to keep the organization on an even keel. The technical presentations were excellent. William Sheffield, the former Governor of Alaska, was our luncheon speaker. He is the individual who was the governor at the time the Alaskan railroad was taken from the FRA and put under the management of the state of Alaska, and he gave a very informative talk. The conference closed out with a train trip between Anchorage and Seward, which is on the coast. We went through very mountainous territory. We stopped at certain areas to look at some of the problems of maintaining the Alaskan railroad that is unique to them which we do not see in our part of the country.

In early October, we were again faced with a decision. Our contract with the AAR for handling our administrative functions entered in 1995 needed to be renewed or renegotiated. At the time that we had divorced ourselves from the AAR, it was felt we could not handle the administrative functions as well as the Executive Director's position all at one time, so we entered into a contract with the AAR for handling administrative functions that worked quite well. In October, we had to make a decision whether we wanted to continue that contract or do something else. Here again, we got the Executive Committee together on a bridge line. I'd just like to reiterate a little bit on the use of bridge lines for board meetings. It was one of the keys to our success this past year involving the board and the Executive Committee in all decisions. At the drop of a hat, we set up the bridge line and told members to call in at a certain time. By doing this, everybody was involved and we had consensus.

In early October, we had a bridge line call to discuss whether we wanted to reenter the contract with the AAR or hire our own staff and do our own work. The Executive Committee recommended that we hire our own office staff and pick the key people since we felt we could better serve the membership and have continuity so things wouldn't fall through the cracks. That was taken to the December board meeting and it had a positive vote. On January 2, Andressa Padden was hired to work for us. She came from the AAR and on January 15, Wendy Tayman was hired to be the Director-Administration. I want to welcome Andressa and Wendy on board. They have gotten off to a good start and here again we see a positive, bright future. Also, we decided to keep the office in Washington with the AAR because we thought that relationship was very important and we wanted to maintain it. The AAR is still doing some basic functions for us, so at this point in time, we decided to stay in Washington.

Another item that we brought up at the December board meeting was to study the constitution—the constitution in general, as well as the membership criteria on committees. We set up two ad hoc committees of five people each. They're made up of past presidents, retired AREA members, one supply person, and one person that's been on the supply side as well as the AREA side. The membership criteria and rules for the guidance of technical committees are being reviewed. We feel that the technical supply people themselves play a very important part on these committees. Here again, it must be handled properly. We must control the voting. Suppliers cannot dominate the sub-committees. We must have professional conduct and we'll probably no doubt have to set up a board membership committee to be an overview to make sure that things are handled properly and that they don't become a social club.

The second one is looking at the constitution in general, which again is very important. There's been very few constitution changes through the years and of course, as I've said before, and I'll say it again today, due to the downsizing and mergers, the Class I railroads do not dominate the AREA like they did in the past. Today, the AREA must serve the industry. The industry today is made up of the short lines, regionals, commuters, transits, and consultants, as well as the supply group. Three years ago, we elected a transit board member who was Jim Palmer. He left the industry and Lynn Wilder fulfilled his term and she was reelected to another three years in this past election. Also in this past election, you'll note that we elected a short line member, Rich Keller from Montana Rail Link. We're in a changing society. We must serve the industry properly, and that's why we're looking at the constitution in general so that this can be done. We feel it's important because down the road, who knows what the board membership might be.

We must look for the other organizations and joint ventures. I was in Anchorage, Alaska for a week. The following week, I'm in Minneapolis at the Roadmasters Conference. That was very difficult for me to support. I'm sure it was very difficult for upper management also to comprehend this. They don't quite understand the relationship as to what each group does. We had a joint meeting there with B&B, Roadmasters, and REMSA to sort of discuss whether there was any merit to having joint conferences and joint ventures to save the industry money, attending meetings as well as better serving the industry. It was decided to set up an ad hoc committee with members from each group to meet and discuss this. The meeting was held recently in Atlanta, which I feel was very positive. Things have to go forward and we're a long way away from doing this because we're locked into Mexico in 1996 and of course, the REMSA show is in Kansas City in 1997. So if we don't get started now, change will never transpire. So we feel this is very important that we do things together—save meeting money as well as serve the needs of the industry.

Other items that we have going on I briefly talked about at the Committee Chairmen's meeting that we must revise the manual change process to better serve the industry and we have a group looking at this. It's moving slow because it's quite an involved process, but we can't give up on this. This must be done.

Also, you've probably seen, or you'll see here at the conference, the electronic conversion of the manual. There's a pilot here today and I'm sure Dave will talk a little bit more about that when he gets up to talk. That will be done this year as well, as we're about 25% done on the CAD portfolio and once the conversion of the manual is done, we'll continue the portfolio. Again, these are steps that are positive.

We have a track buckling seminar set up in the East with Alan Zarembski primarily aimed at the transits and commuter railroads that don't have programs. We also have a seminar set up in August here in Chicago on turnouts and track work. That's one of the most expensive items for the Class I's, as well as all the railroads in general. So here again, we feel these are needed in the industry as well as they'll generate revenue for the Association.

The next item I'm going to talk about I'm sure has caused a great deal of controversy. There's been a lot of gnashing of teeth and that is the reason for the conference fee increase. It's a twofold purpose. Number one is we now have our own staff and if we want to properly serve the membership, Committee Chairmen, and the industry, we need a well qualified staff and we must pay them proper wages as well as proper fringe benefits. Since we're now on our own, the fringe benefits package is much more costly than it was in the past. So our costs to operate have increased due to being a small association. So that is one reason and of course, the second reason is the three-day conference of this type. If you look at other societies like ours or other associations, our past fee was a sort of a "Monday morning special" and it was very cheap for what you received, so we feel that we had to raise it to get more in line with what you were getting versus other associations, and we still think that we're fairly economical when you compare us to the value that you get from these conferences. So those are the two reasons for the increase.

Another item just in passing is the AAR/AREA relationship that we feel is important, particularly with our committees, and I'd say we must sing out of the same hymn book because the work that our committees do must not contradict what the AAR technical people in research are doing. We can't publish a paper saying one thing and AAR publish something else because we would confuse the industry. We have communications along those lines to resolve those issues which I feel is most important.

A long range problem within the industry is young engineers. That's not only an industry problem, it's also an AREA problem. Due to all the downsizing and mergers, we're just not bringing young engineers in at the bottom, yet we're surviving today with consultants. But somewhere down the road the consultants are going to lose the people that they've picked up from railroads from buyouts, mergers, and so on and there's going to be a definite need. Now I certainly don't have the answer, but it's something we have to address. We must continually address it with upper management to resolve the issues because I feel it's critical.

In summation of the year, I had goals when I took over a year ago. Of course, the number one goal was to hire a new permanent Executive Director, which was accomplished. I hired Dave Staplin, who I think has a very bright and positive future. He has the required vision to lead the association into the future. He's already come up with many good ideas, good plans to lead us into the future and I see nothing but positive direction since Dave took over. And of course, the other item was to keep the association on an even keel to serve the needs of the membership and the Committee Chairmen, and that was done. We had a successful conference in Anchorage. We had successful Committee Chairmen's meetings. As a matter of fact, we've had two. We've had an active board. As I said earlier, we've had many board meetings, many bridge line calls. When something started to get out of hand, we'd set up a bridge line call, whether it was an evening, Saturdays, whenever, and discuss the matter and arrive at consensus and continue on our way. I believe that is probably one of the key points to keeping the association on an even keel this past year, moving forward as we did.

I want to thank everyone—the membership, the Committee Chairmen, and particularly the board—for all the help and participation and patience they gave me. This is probably the most active board that we've ever had and we had no choice but to be active the way we were operating and some of the problems that came up, so here again, I want to thank them. I said a year ago that I saw nothing but a bright and positive future and I have not changed my opinion one bit. We had a positive year and again, I see a positive future. Thank you very much.


R
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TIMBER TRUCKING

DEVELOPMENT OF THE RAILWAYS IN GERMANY AFTER REUNIFICATION

By: Prof. Theo Rahn*

Introduction

When I was asked to give a presentation on the development of the railways in Germany after reunification, I considered the difference in size between Germany and the United States. I thought whether it would not appear to Americans as if I were talking about the merging of Disney World and Disneyland? However, after a moment's thought, it became clear to me that much of the experience gained from the developments in our country would also be of interest to you here in the U.S.A., in particular as Pan-European developments are concerned.

At the end of 1989, communism in Eastern Europe began to collapse. Thanks to the consent of the powers of the Second World War, and in particular with the support of the government of the United States of America, Germany was able to regain its national unity with the unification of the two German states—the Federal Republic of Germany and East Germany. Formally, reunification meant that East Germany joined the Federal Republic of Germany. The new and larger Federal Republic of Germany from then on had *two* railways which were developed very differently during the more than 40 years of separation (Figure 1).

While the West German Railways [Deutsche Bundesbahn (DB)] in the West German territory was subjected to strong competition from road and river transport due to the market economy, the East German Railways [Deutsche Reichsbahn (DR)] was an instrument of the planned economy of the complete Eastern Block and was in a very bad state (Figure 2).

The country, the people, and also the railway network were separated for more than 40 years by the Wall and the Death Strip through the middle of Berlin and the middle of Germany (Figures 3, 4, 5, and 6).

I have divided my presentation into several sections. First, I would like to discuss the situation of the two German Federal railways before unification. In this context, the starting data for the two German railways in relation to size of network, length of line operated, volume of traffic, turnover, economic results, and investment might be of interest.

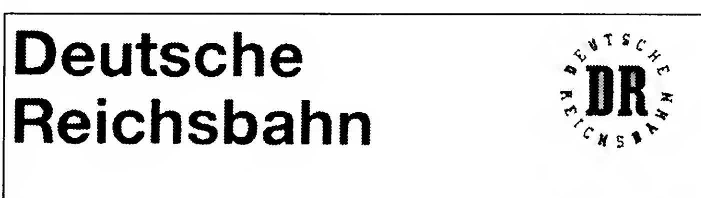

Figure 1.

*Chairman, Managing Board of Deutsche Eisenbahn-Consulting, Frankfurt, Germany

Figure 2.

Figure 3.

Figure 4.

Figure 5.

Figure 6.

Secondly, I shall describe the measures which were and are still necessary to merge the two systems and to adapt them to market requirements. Mention will be made of the "Gap Closing Program" and the "German Unity Transport Projects." I shall use the newly-built Hanover-Berlin line as an example for long distance services and the reconstruction work in Berlin as an example for commuter services.

The development of railways in Germany must be seen in a European context. The opening of the countries of Eastern Europe and the growing together of Europe means that Germany cannot be viewed as an island. Germany is the most important country of transit where large European traffic flows cross. If the measures in Europe are not viewed in isolation, but from a European perspective, the dimensions are more comparable to American dimensions.

Starting Data of the Two German Railway Companies Before the Formation of the Joint Company Deutsche Bahn AG

Following the reunification of Germany, the Deutsche Reichsbahn (DR) and the Deutsche Bundesbahn (DB) were two legally separate, state operated companies for the period from 1990 to 1993. The Deutsche Reichsbahn was the railway company of the former East Germany, the Deutsche Bundesbahn operated in West Germany. Both railways had passenger and freight services.

The Deutsche Bahn AG (German Railway Company) was founded on January 1, 1994. Thus, the two federal railways were converted into one enterprise with private sector structure.

Table 1 shows the starting situation of the two German railways in 1993.

Approximately 50% of the networks are electrified. In staff training and other fields such as safety-philosophy, no significant differences exist.

Measures for the Upgrading of the Railway Infrastructure Plus New Construction in the Course of German Reunification

The equalization of the living standards in East and West Germany was a political priority target of German reunification. To achieve this target, considerable sums were transferred from West to East Germany. A large part of these funds flowed directly into private consumption. Even today, about 100 billion D Marks are transferred annually from West to East Germany into the budgets of the "New Federal States."

However, it was recognized politically at an early date that the adjustment of living standards could only be achieved permanently by improving the infrastructure. A modern infrastructure had to form the basis for independent economic growth in East Germany. For this purpose, particular emphasis was placed on the railways as carriers.

Table 1.

	DB	DR
Personnel	218,000	138,000
Length of Route (km)	26,000	14,000
Length of Track (km)	52,000	25,000
Switch Points	86,000	57,000
Tons of Freight (Million)	231	83
Tons—Kilometers (Billion/km)	52	13
Passengers Carried (Billion/km)	1,108	326
Passenger—Kilometer (Billion/km)	47	10
Investment (Billion DM)	5.1	6.4
Operating Loss (Billion DM)	–5.8	–5.3

As a first step, the gaps caused by the division of the two countries had to be closed in order to meet the requirements of freight and passenger transfer within Germany (Figures 7, 8).

The Gap Closing Program

The German Secretary of State for Transport initiated the "Gap Closing Program" in 1990 jointly with the East and West German railways. The program's objective was to link the lines which had been separated due to the division of Germany. Depending on the situation, the measures involved the reconstruction of links, the extension to double lines, and electrification. A total of nine (9) projects was estimated at 6.4 billion D Marks.

For economic reasons, it was not intended to re-establish all railway links between East and West Germany which had existed in 1945. Rather, the efficiency of the main axis was to be guaranteed. This decision quickly proved to be correct. Even before completion of these measures, a considerable increase in freight and passenger traffic was achieved on the main axis (Figure 9).

"German Unity Transport Projects"

The first step in linking the transport networks in East and West was followed by the "German Unity Transport Projects." As early as April 1991, the German government decided on seventeen (17) individual measures, nine (9) of these being rail projects. By the year 2000, all important East-West links are to be modernized. The nine rail projects are valued at about 30 billion D Marks. This could be realized up to now without any restriction as far as the rail projects in West Germany are concerned.

The "German Unity Transport Projects" had a key function in knitting together the old and the new German Federal states. And from this key function, the political task of financing the infrastructure measures was developed. The following photographs show the situation before and after the improvement measures (Figures 10, 11, and 12).

German Transport Plan 1992

In 1992, the "German Unity Transport Projects" resulted in the first overall German transport master plan. The rail network was allotted the largest share of total expenditure, 195 billion Marks and 40% of the total up to the year 2010.

For the railways, the emphasis in investment policy is placed on basic modernization and upgrading of the transport infrastructure in the area of the five new Federal states, construction of a high-speed rail network in Germany and Europe, and reduction of capacity bottlenecks in the rail network.

Figure 7.

Figure 8.

Figure 9.

Figure 10.

Figure 11.

Figure 12.

Financing takes place via the German government budget. The capital invested is repaid by the users via track charges. These are user fees which are based on depreciation, interest on the capital invested, and maintenance costs of the infrastructure. This method has proven itself to be successful and has led to fast implementation of the necessary infrastructure measures between East and West Germany. There is very little interest from the private sector in railway construction due to the almost complete decline in the East German and East European economies.

The "Gap Closing Program" is now practically completed. The "German Unity Transport Projects" are partially completed and are now in an advanced stage of implementation as, for example, the construction of a new line between Hanover and Berlin.

The New Hanover-Berlin Line

After the Second World War, the two Hanover-Berlin rail lines in the area of the then Soviet occupied zone were reduced to single track lines. However, both lines played a prominent role in securing the Allied land access to Berlin.

Up to 16 freight trains per day secured the supply of Berlin.

By reinstating a double track on the southern line after reunification, the transport importance of the Hanover-Berlin corridor increased significantly (Figure 13). This led to the decision of constructing a high-speed link between Hanover and Berlin as part of the European high-speed rail network. As of this date, approximately 120 trains travel each way over the Hanover-Berlin corridor. Forty percent of these trains are freight trains (Figure 14).

The measures necessary for the efficient upgrading of this line for passenger and freight traffic and for increasing the speed up to 280 km/h (175 mph) were as follows:

- track improvement (Figure 15)
- electrification (Figure 16)
- removal of all crossings (Figure 17)
- new signaling system
- installation of continuous speed control for automatic train operation with high speed

Modern, high-performance machines made an important contribution to cost savings and quality improvements. For this purpose, the Plasser SVV-100 Sand and Ballast Distribution and Compaction Machine was developed (Figure 18).

The high performance RM-800 was used for the continuous removal of ballast and soil. To save costs, the ballast is removed separately, processed on the spot, and re-installed as sub-ballast. This saves on expensive raw materials and high transport costs (Figure 19).

An extremely high construction performance, with up to 350 m/h, can be attained with these machines and very precise installation tolerances can be achieved. For the upgraded part of the new Hanover-Berlin line, about 25,000 meters of substructure and ballast formation were produced in only three months (Figure 20).

The considerable performance of the SVV-100 and the RM-800 high-performance machines, in conjunction with the other high-performance machines such as the stabilizer, is an important factor for the completion of the Hanover-Berlin project on time in 1997 (Figure 21).

Berlin Projects

After reunification, Berlin became the capital of Germany.

At present, Berlin looks like a construction site. Nowhere else in Europe are so many infrastructure projects being planned and implemented (Figure 22).

Figure 13.

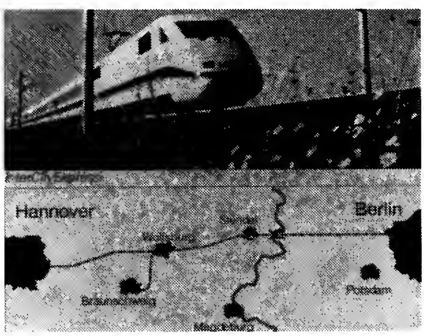

Figure 14.

Figure 15.

Figure 16.

Figure 17.

Figure 18.

Figure 19.

Figure 20.

Figure 21.

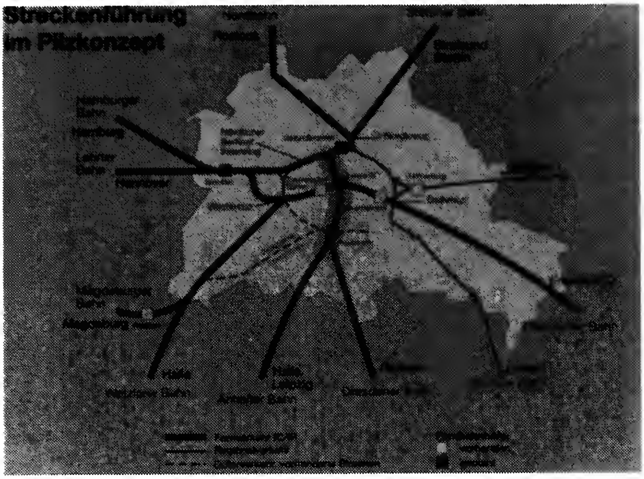

Figure 22.

For the railway system, a new concept was suggested which provides for a tunnel right through downtown Berlin consisting of two (2) commuter railway tracks, two (2) long distance railway tracks, and a 4-lane urban expressway connection. The tunnel length is about 6 km (3.75 miles). The costs are estimated at about 2 billion D Marks. In addition, the complete existing rail system has to be renewed (Figure 23).

I would like to introduce the Berlin Zoo-Berlin Central Station at this point. The section should be of particular interest as it runs on a viaduct with commuter and long-distance tracks running in parallel (Figure 24).

The viaduct comprises 530 viaduct arches, 60 bridges, four commuter railway stations, and five stations for long distance. The long distance and commuter railways run side by side on two tracks, each on the local viaduct (Figure 25).

Tasks for modernization are:

- modernization of the complete superstructure for commuter and long distance railways (Figure 26)
- upgrading of the platforms in the Berlin Central and Berlin Zoo stations allow usage of Intra City Express (ICE) high-speed trains
- use of modern signaling equipment
- electrification of the complete line

The construction measures are taking place largely "under the rolling wheel" and will be completed by 1997.

During the first half of the construction period, the commuter railway used the long distance track; during the second half, the commuter railway will be using its two modernized original tracks.

The target date to bring the ICE high-speed train to Berlin Central Station by 1997 appears to be on track.

Figure 23.

Figure 24.

Figure 25.

Figure 26.

As a result of this construction work, a structure, which was divided into East and West for more than 40 years, will once again shape a unified Berlin cityscape. The measures mentioned:

- the "Gap Closing Program,"
- the "German Unity Projects," and
- the railway concept for Berlin,

are all part of the program to lead Deutsche Bahn AG (the German Railway) to a prosperous future.

By changing the German constitution and 32 laws, the "Deutsche Bahn AG" was founded from the two German Federal railways, DB and DR, on January 1, 1994. At present, the German state is still owner of the privatized Deutsche Bahn AG. But, the next step, when

- a track company,
- a long distance railway company, and
- a commuter railway company,

are created from Deutsche Bahn AG by 1997, these companies will be listed separately on the stock exchange. The success of the measures taken already became apparent after the first year, 1994. Table 2 shows the most important data from 1994:

Table 2.

	DBAG
Personnel	327,000
Income (Million DM)	29,000
Expenses (Million DM)	28,800
Annual Surplus (Million DM)	180

The Germany railway system is looking good for the future.

A SUMMARY OF SEVEN YEARS OF RAILWAY BRIDGE TESTING AT CANADIAN NATIONAL RAILWAY

By: Robert A. P. Sweeney,* George Oommen, and Hoat Le*****

Abstract

This paper provides a summary of field measured static (crawl speed) point stresses compared to theoretical calculated stresses (as used in normal bridge rating practice), together with impact values compared to the AREA Manual equations given in Chapter 15, all under controlled test trains. Several changes to the current AREA Manual are suggested.

Introduction

Market opportunities are encouraging the railroads to consider the acceptance of higher axle loads on a selective basis. As this trend continues, all or most of the mainlines of the major railroads will be permitting 286,000 lb. cars on 4 axles and 420,000 lb. locomotives on 6 axles relatively soon. In spite of increased anticipated infrastructure costs, the net benefit of 286,000 lb. loading appears to be positive for many railroads and their customers.[1]

In anticipation of increased axle loads, Canadian National Railway's (CN's) Engineering Department has undertaken a major bridge testing program since 1988 as an adjunct to its rating program. The purpose of this program is to ensure the safety of its aging bridge plant, to prolong its life, and to prioritize replacement and strengthening programs. This paper summarizes and reviews the results of 70 full scale field tests using controlled work train consists on steel bridges. The data presented is based on maximum recorded point stresses.

Bridge Testing Program at CN

CN had a five year program to load test major bridges starting in 1975, and decided to add bridge testing to the evaluation process for smaller bridges in 1988. In both cases, the motivation was to save or at least substantially delay major capital expenditures. From this point of view both programs have been a resounding success.

Testing

A standard test[2] consists of three parts:

- Recording of regular traffic for statistical data (not covered in this paper).
- Static loading of the bridge to determine the "Alpha" factor (ratio of measured to theoretical stresses). This paper presents this data in terms of the maximum measured point stresses which is valid for critical fatigue determination, but not necessarily for other applications.
- Dynamic loading of the bridge to determine the impact at various speeds, again based on point stresses.

In most cases, the Alpha factor can be used to enhance the capacity rating, and the statistical and impact data can be used to predict the life expectancy of certain components, resulting in great savings in operating and capital expenditures.

Static and dynamic effects were measured using a pre-weighed work train under controlled conditions. Generally, the work train consisted of one or two locomotives followed by six or more cars

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** System Engineer Bridge Assessment, Canadian National Railway
*** Bridge Testing Engineer, Canadian National Railway
[1]See Hargrove et al., 1995.
[2]For details of CN's Testing Equipment and procedures, see Oommen and Sweeney, 1992.

fully loaded, and were sometimes followed by three empty cars. The tests were conducted at various speeds ranging from crawl speed to a maximum of 70 mph for freight trains and 110 mph for passenger trains. Maximum allowable speed varied depending on the zone speed of the line.

Selection of Spans for Bridge Testing

The basic concept of bridge rating and safe life evaluation used by CN's Bridge department is a multiple step procedure varying from a simple check against provisions similar to those contained in Chapter 15 of the AREA Manual,[3] to a full scale load testing and crack evaluation.

The first step involves checking critical details against the design provisions of the Manual. If it is adequate, no further action is warranted

Next, a detailed analytical evaluation is made using the approved rating and fatigue procedures. If the span and detail in question passes this test, no further action is warranted.

If the previous steps reveal structural inadequacies, and the cost of replacement or repair is high compared to the cost of a successful load test, the structure is then load tested. Line importance also plays a major role in the selection of bridges for testing.

Description of Bridges/Spans Tested

Between 1988 and 1995, CN's Bridge department has carried out over 70 field tests. The majority of the bridges tested were on the main line supporting traffic in the range of up to 70 MGTM. Most of the traffic in Eastern Canada is of mixed type while most of the traffic in Western Canada is of unit trains.

The tests were conducted on various types of spans, a majority of which were built around the turn of the century. Included in these tests were 28 through truss spans, 13 deck truss spans, 1 beam span, 6 through plate girder spans, and 22 deck plate girder spans.

The truss spans investigated were of riveted construction. Generally, the construction was typical of turn of the century designs. The top chords and compression members were built-up sections while the bottom chords and other tension members were either built-up members or eye bars with or without pin plates.

All the plate girder spans were of riveted construction except one welded span, and the beam span was built using rolled I-beams.

Decks were generally open deck timber. There were five ballasted decks and three steel plate decks. The rails were generally 136 lb. continuous welded rails on heavy tonnage lines with or without "Conley" expansion joints to 115 lb. jointed rails on the low tonnage lines.

The substructures consisted of stone masonry and concrete piers and tower and pile bents. Conditions of the bearings ranged from satisfactory to poor. In order to simulate every day field conditions, approaches were not surfaced or tamped for the tests.

Alpha Factor

The alpha factor is defined as the ratio of the field measured static live load stress to the theoretical static live load stress. Since there is no built-in safeguard against unintentional errors in testing and theoretical stresses are computed according to the rating guidelines, which do not necessarily reflect true boundary conditions, caution should be applied when using this factor for bridge rating and predicting the remaining life. Since the alpha factor is subject to the interpretation of the engineer,[4] it is a good practice to set a limit of 0.8 unless a more appropriate theoretical model is developed to theoretically explain the resulting behavior.

One obvious example is the case of a simple beam or girder span with frozen bearings which has an effect on the alpha factor. The rating engineer must consider this and other factors in interpreting the test results.[5]

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[3]See American Railway Engineering Association Manual of Recommended Practice.

[4]See Oommen and Sweeney, 1992.

[5]See the three references by Bahkt and Jager.

Discussion of the Results

The field measured stresses were compared with the theoretical stresses calculated based on simple analytical models (as used in normal bridge rating practice). All of the data was taken at temperatures above the freezing point (32°F or 0°C). The measured data are in the raw format without any adjustments. The measured stresses do not include dead load and are typical of the live load stress ranges that cause fatigue damage in North American railway bridges. The maximum measured static stress was 11 ksi in a stringer. Typical static stresses in these tests were in the range of 3.5 to 7.0 ksi, except for stringers and girders which were typically 3 to 10 ksi.

Figures 1 through 11 contain static point stresses (crawl speed) from work trains only. Figures 12 through 16 show impacts from the same work trains. The results of stress ranges from regular traffic are not included in this paper. Since the main goal of this paper is to present the actual results as is, no attempt was made to justify the results using advanced analytical techniques. In many cases this information is available in CN internal reports.

The range of loaded lengths of the members tested are shown on each of the figures for stresses. All plotted values are the maximum values recorded and do not represent the average cross-sectional stresses nor are the effects of bending, torsion, or axial loading sorted out.

Truss Spans

The measured static stresses of the top chords (see Figure 1) ranged between 66% and 92% of the calculated stresses except in one case where the measured stress was 9% higher than the theoretical value. This structure is a through truss supporting two tracks in the middle with cantilever brackets supporting a two lane roadway on each side. Note that all but three exceed the alpha of 0.7 given in part 9 of the Manual.

Figure 2 shows the comparison of measured static stresses and theoretical stresses of bottom chords of the truss spans. Aside from one case, all the measured stresses were lower than the theoretical stresses. They varied from 42% to 96% of the theoretical value. The exception was a double track pin connected "fish belly" deck truss.

The measured static stresses in the hangers (inside face) were consistently higher than the calculated axial stresses (see Figure 3). The measured stresses were between 102% and 131%[6] of the theoretical stresses (generally, out of plane bending is not considered in the rating of hangers). The two exceptions below theoretical were 87% (gauges located near the top of hanger where shear lag may have been a major factor), and 77% (cantilever bridge with suspended spans that contains articulated universal joints that preclude anything but axial force transmission). In this case all but one exceeds the alpha of 0.85 given in the Manual.

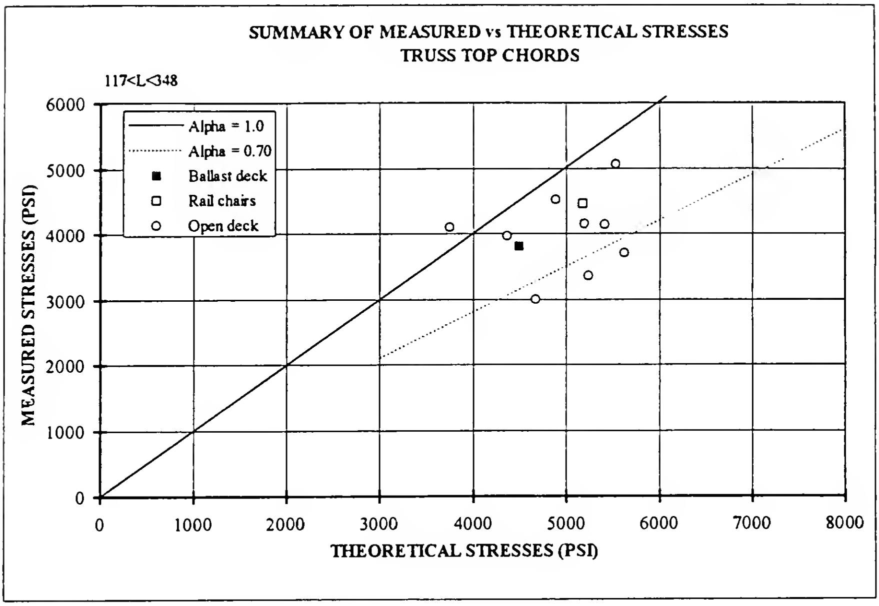
Figure 4 shows that the measured static stresses in the end posts were between 87% and 128% of the calculated value which is a function of the boundary conditions and relative stiffness of the post and the chords. In one case the alpha was only 56%, whereas the rest exceed the value given in the Manual.

The measured static stresses in the web members were between 208% (where all the eyebars were not sharing the loads equally) to 50% (counters were not participating fully) (see Figure 5).

Stringers and Floor Beams of Trusses and TPG Spans

The alpha factor for stringers was generally less than 100%, however in a few cases, the measured stresses were higher than calculated values. Top flanges had values between 38% and 171%. The higher values are due in part to the fact that in some cases the ties were not resting properly on some of the stringers in a multiple stringer configuration. Hence, one stringer would record a low value and another a high value. The range for the bottom flange was 32% to 135% (see Figures 6 and 7).

[6]A previous set of tests showed up to 152%. See Fisher and Daniels, 1976.



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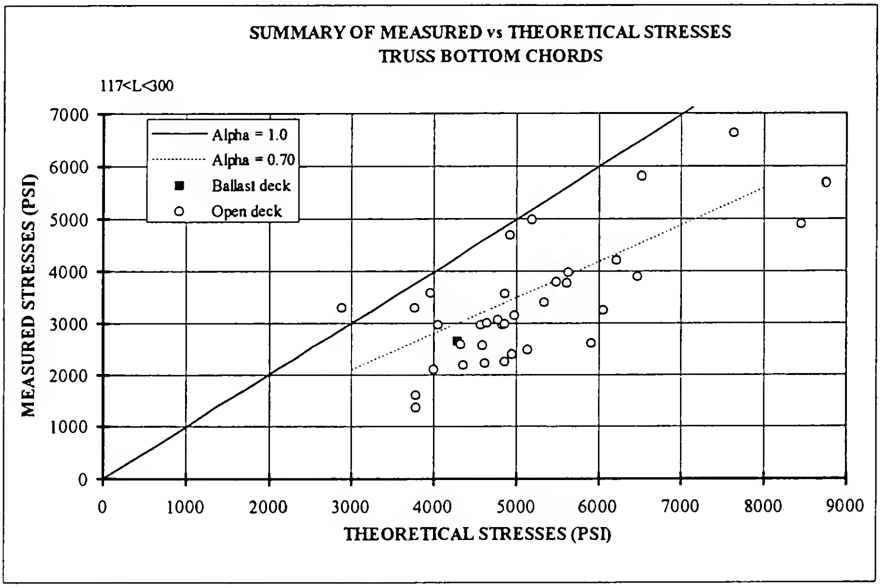

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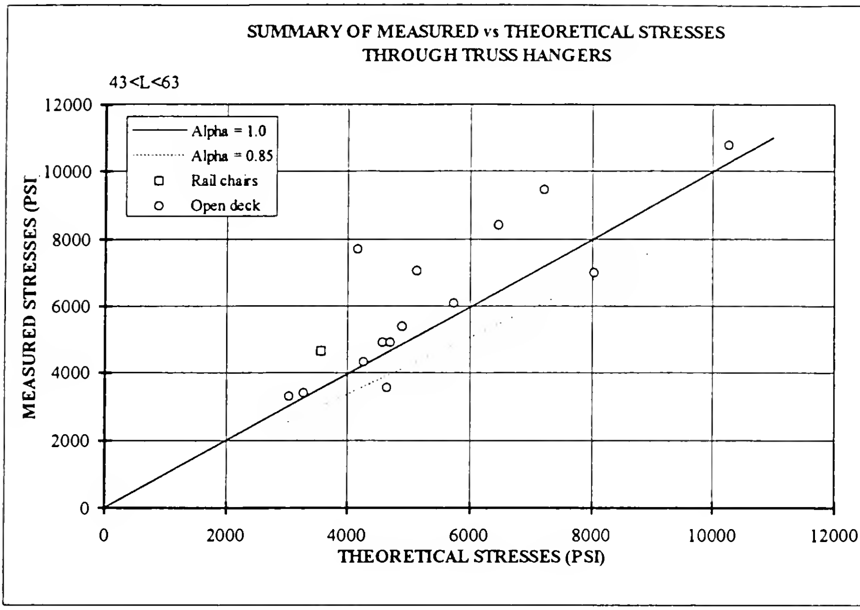

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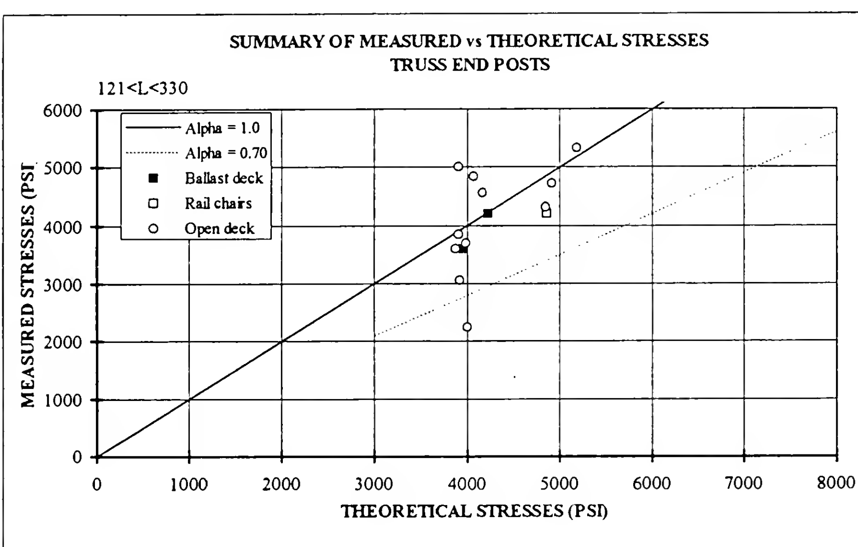

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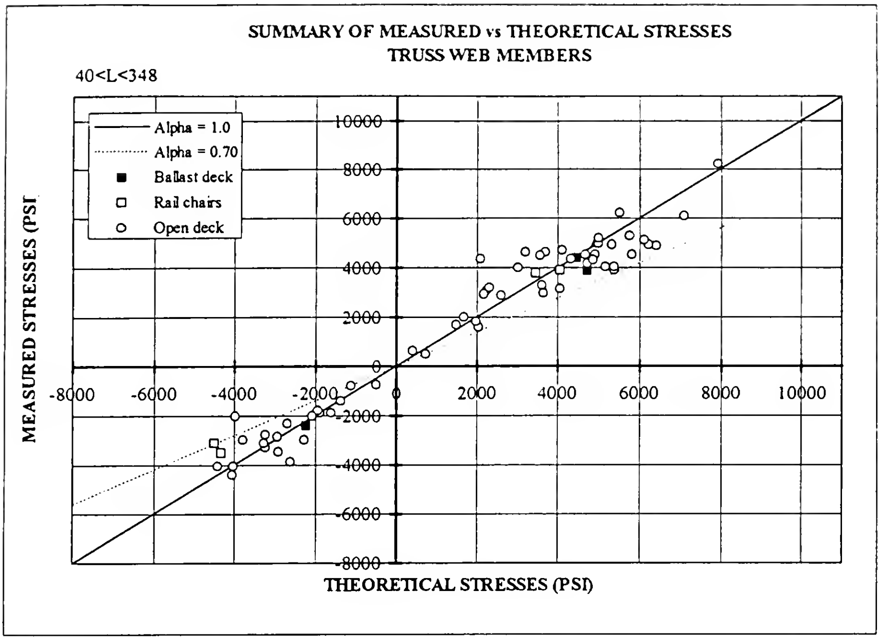

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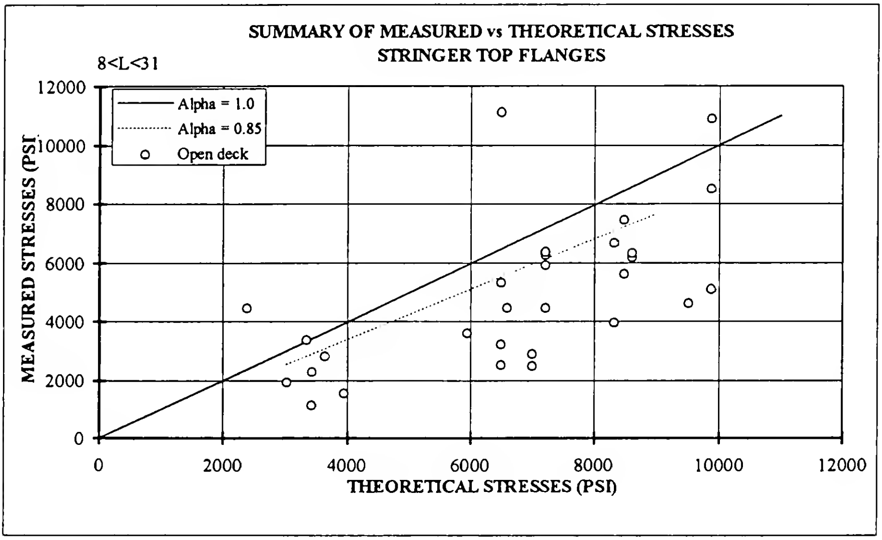

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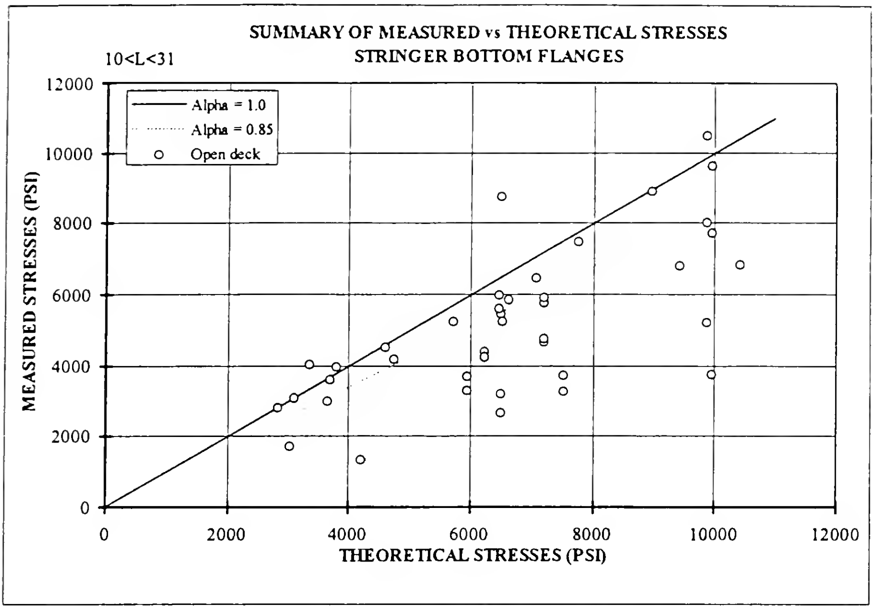


Figure 7.

Figures 8 and 9 show that the static stresses in the flanges of the floor beam, in most of the cases, were less than the calculated stresses. Alpha factors for top flanges varied between 52% and 112%; bottom flanges varied between 43% and 113%. Generally, the higher stresses were due to local conditions such as pitting, reduced section, etc.

Girder Spans

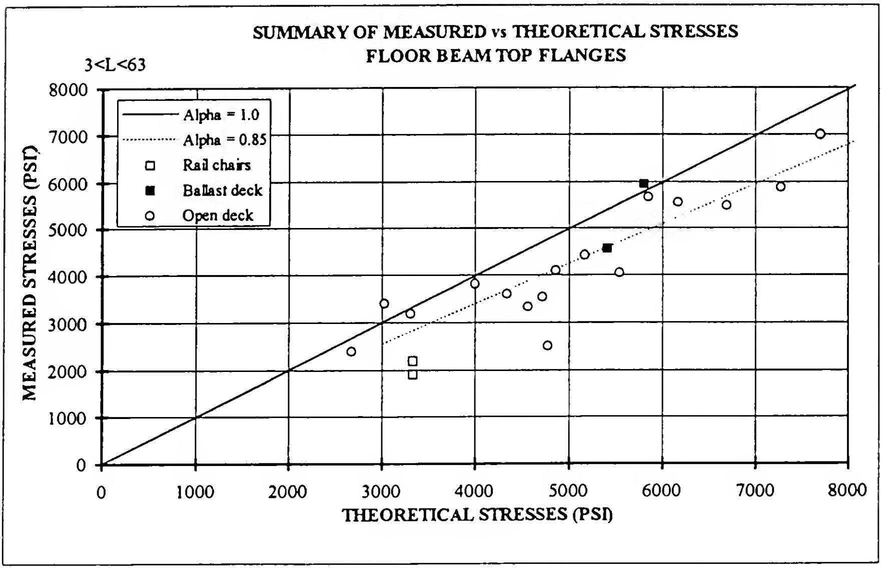
Measured girder static stresses were compared with the theoretical stresses and were plotted on Figures 10 and 11. The alpha factor for top flanges varied between 22% and 132%. There is one low value of 3% showing composite action with an encased concrete deck. An unusually high number of top flanges were supporting higher stresses than the calculated values. One explanation is the out of plane bending due to ties. The bottom flanges varied between 34% and 111%.

Impacts

Figures 12 through 16 show the relationship between theoretical impact (as defined by the AREA Manual, Chapter 15) and measured maximum impact (based on maximum measured point stresses). A dotted line indicates the impact recommended by AREA for fatigue calculations on spans greater than 30 feet. According to the field test data, maximum measured impact did not always occur at maximum speed.

Measured impact for truss members, except hangers, were generally less than 65% of the theoretical calculated impact (Figure 12). However, higher impacts were recorded at a few locations, often in the first or second panels of the bottom chord where measured values were influenced by transition effects such as rail joints and stiffness changes as the train comes onto or off of the bridge. All of the higher impacts were on open deck spans. The highest recorded impact was 142% of the calculated impact.

Maximum measured impact for hangers was 42% of the calculated impact, again an open deck span (see Figure 13).



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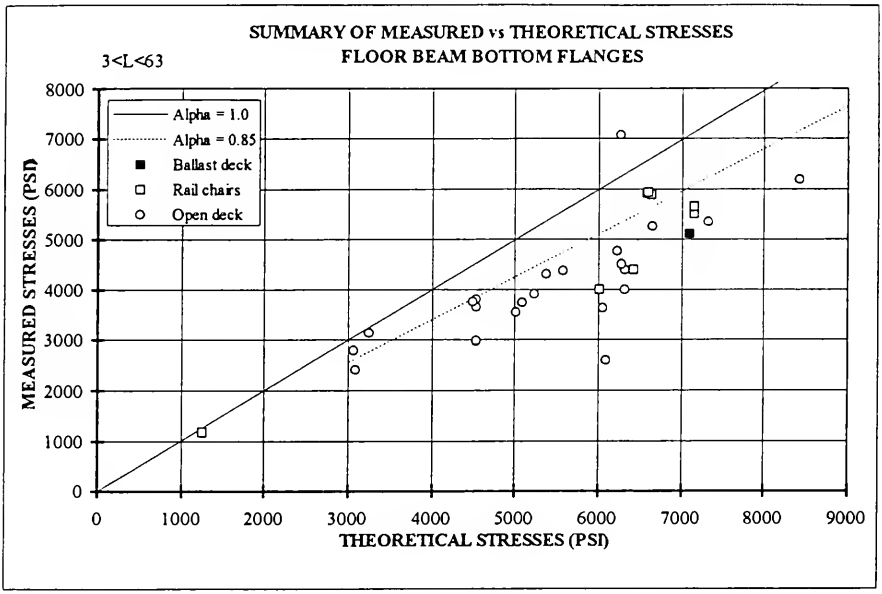

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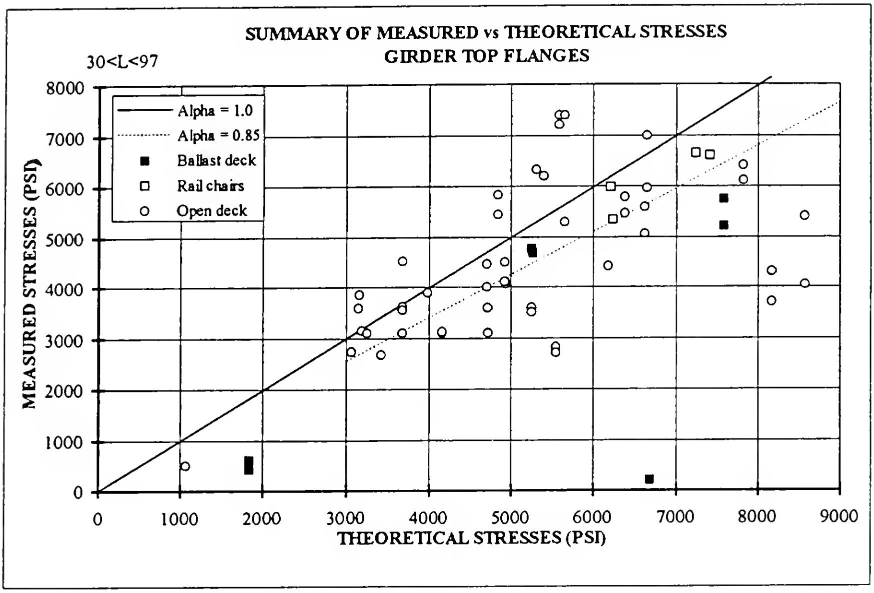

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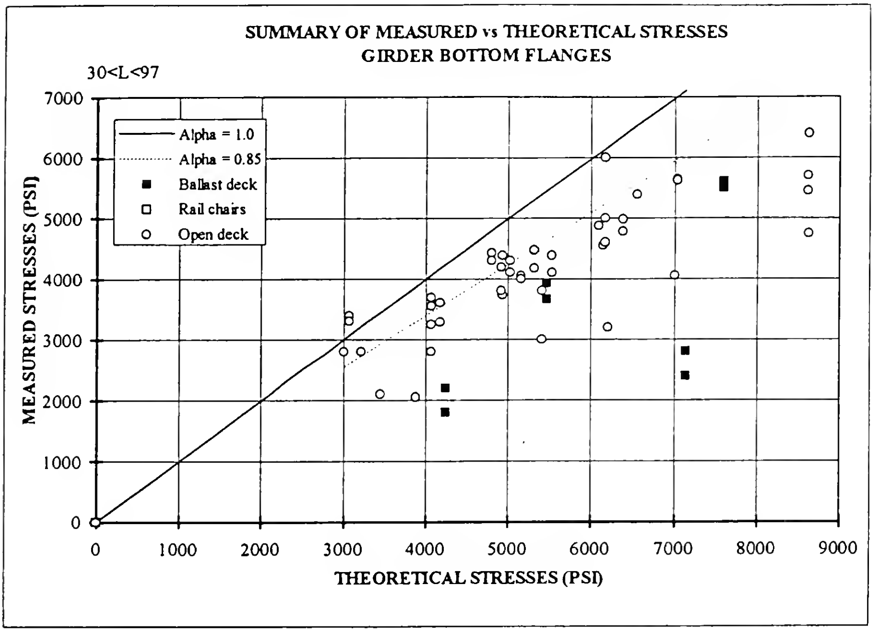

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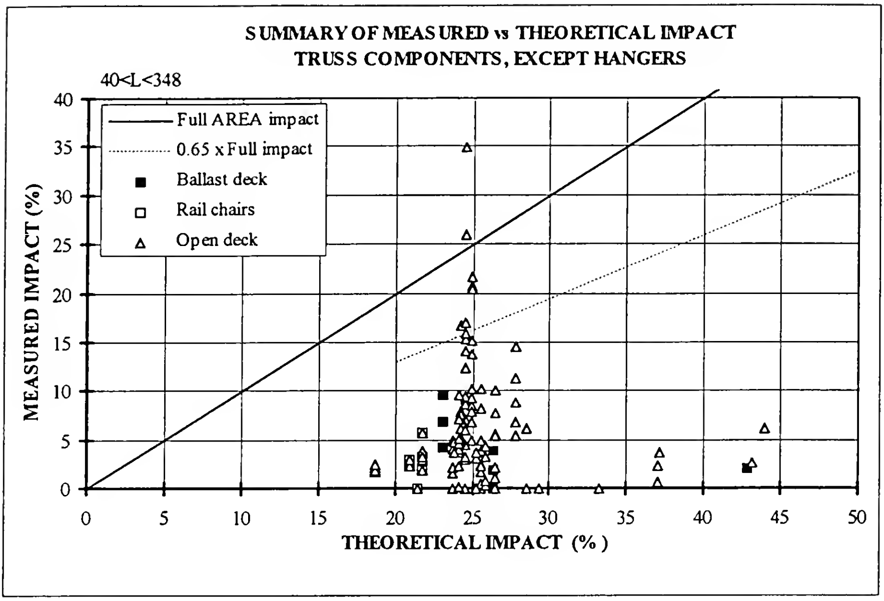


Figure 12.

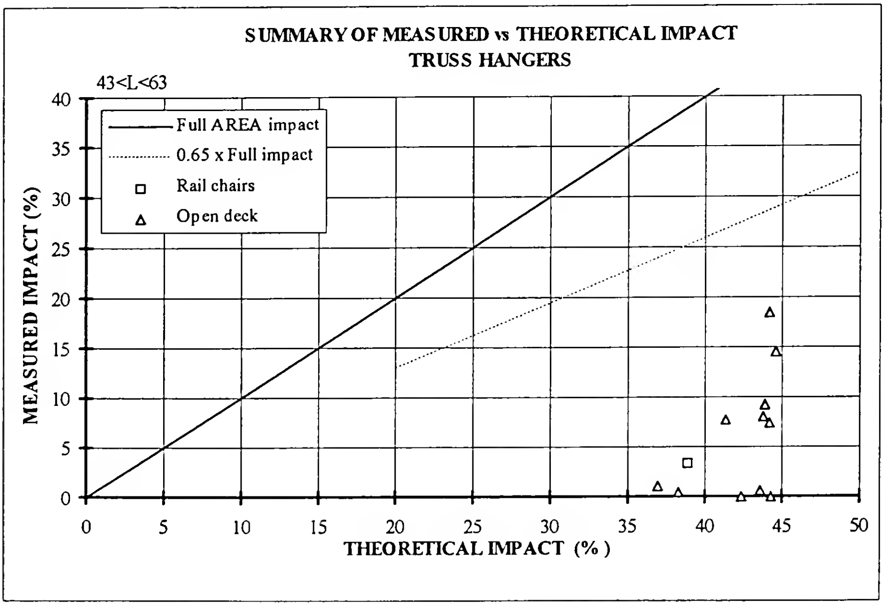


Figure 13.

Figure 14 shows that measured impacts for stringers were below 30% of the calculated impact except at three locations where rail joints were in poor condition. The worst of these was 49% of the calculated impact. All were on open decks. Recorded maximum impact for the floor beam was 41% of the calculated impact which was on a steel plate deck span (see Figure 15).

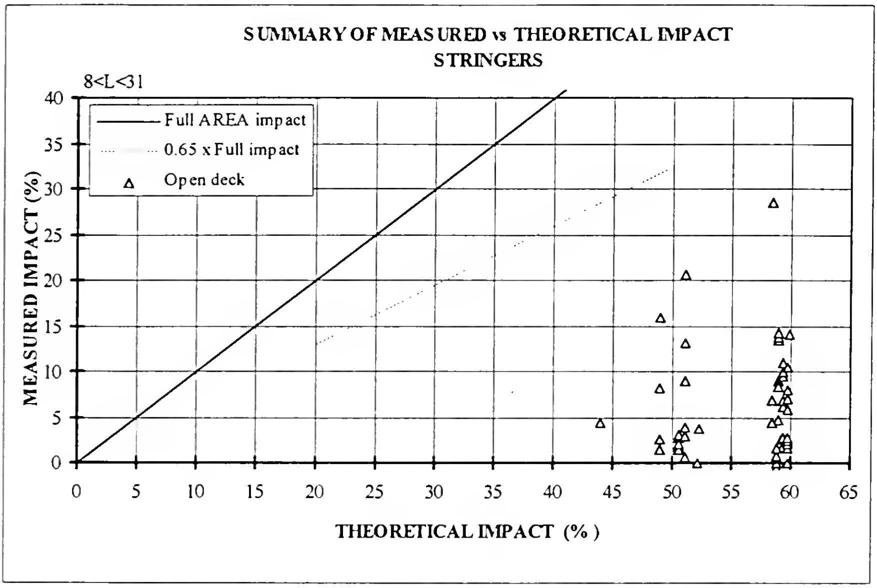
The maximum measured impact for girder spans was 44% of the theoretical impact again on an open deck span (see Figure 16).

Since most of the tests were carried out on CN's mainline, the track was generally in good to excellent condition. Given that the maximum values for impact covered by the AREA impact formula are for rare events occasioned by impulsive loads due to flat wheels or battered jointed or welded rail joints, the relatively low impact values recorded are to be expected. It is our opinion that the impacts measured, mirror typical geometry of approaches on CN mainline track in terms of induced bounce, roll, pitch, yaw, and twist and include relatively few impulsive loads.

Conclusion

Generally, field measured static stresses and impacts outlined in this paper are lower than stresses calculated using conventional analytical techniques. In some cases remedial measures can be delayed for long periods of time. Even when a member is overstressed, testing will often point the way to less expensive retrofits, repairs, or strengthening. Hence, in the majority of cases, bridge testing saves money.

Nevertheless, since the test data shows that there are many exceptions, it is not recommended to blindly assume that such is always the case. The AREA Manual, Table 9.1.3.13A of Chapter 15 has alpha factors which are not consistent with the data presented here. The data plotted in this paper are point maximums, generally based on open deck structures, not cross-sectional average stresses based on decked highway bridges. Committee 15 should discuss which characteristic is more appropriate for their purpose, and the profession needs to be aware of the difference. The resulting conclusions drawn from each will be considerably different.



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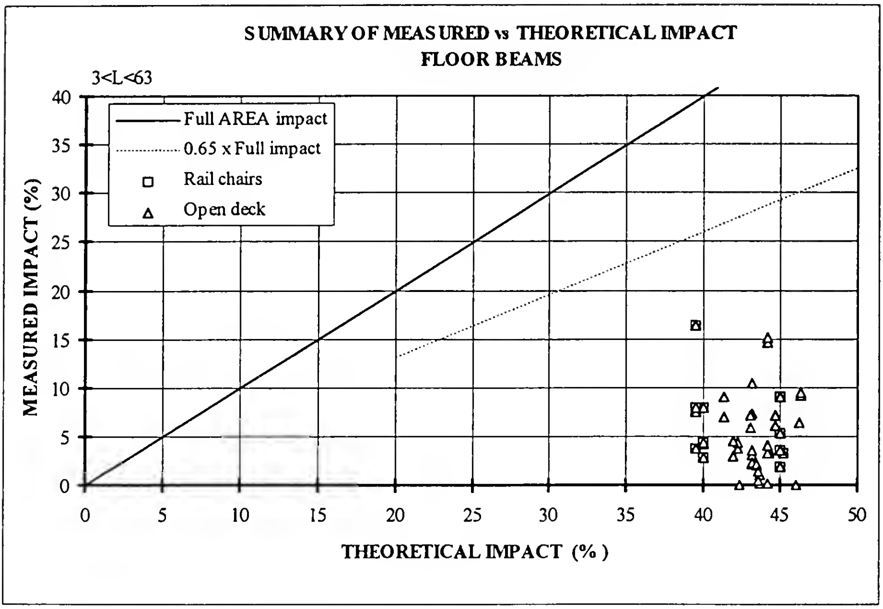

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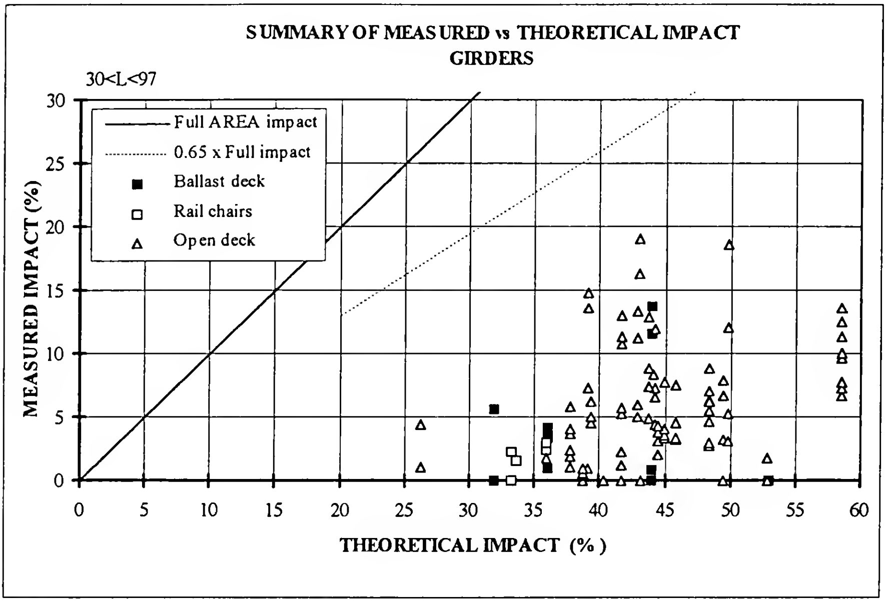

Figure 16.

In evaluating hangers, the effect of bending, both in-plane and out of plane should be included, as should the effects of shear lag in those members which don't have all component parts of the member connected through the joint. Chapter 15 does not adequately cover these items at present.

Clause 7.3.4.3 (e) of Chapter 15, which ignores certain secondary stresses although appropriate for Maximum Rating, is not appropriate when dealing with a fatigue calculation due to the lower levels of live load and impact stress ranges that can cause damage.

Impact factors calculated according to the AREA Manual, Chapter 15 are always conservative, except when the deck and track surface are poor or where wheel maintenance is low. Hence, a lower impact factor may be used for fatigue evaluations. The AREA Manual, Article 1.3.13 (d) of Chapter 15, uses a reduction factor that is more conservative than the data presented in this paper would suggest with the exception of the lower cord of some trusses that were highly influenced by bad rail joints and the roadbed to truss transition. A caution should be added to the commentary, that the reduction in impact for fatigue should not be used where the stress at the detail being evaluated can be highly influenced by a known bad rail defect, or where wheel maintenance practice is known to be poor. Otherwise the Manual's recommendation on this matter seems appropriate.

Where test results are used to modify the capacity rating of components, a safety margin must be introduced to take into account possible systemic, systematic, and human errors.

Acknowledgments

The authors wish to thank Romel Scorteanu, Project Engineer, who is responsible for the data acquisition which he contributed to us, present and former members of the Rating and Testing groups at CN, and R.W. Richardson, Chief Engineer, Canadian National Railway (CN) for his continued support.

Disclaimer

The data provided in this report are of a highly technical nature and greatly summarized. Users need to be extremely cautious in its use.

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[7]Contact IHHA Inc., c/o: 19 Holloway Dr., Lake St. Louis, Mo. 63367-1357, U.S.A.

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DEVELOPMENT OF RAIL GAGE FACE ANGLE STANDARDS TO PREVENT WHEEL CLIMB DERAILMENTS

By: Allan M. Zarembski, Ph.D., P.E.*

Introduction

Rail represents that part of the track structure that first "meets" the wheel and thus directly carries the wheel/rail loading imposed by the traffic operating over that track. As such, it is subject to a significant level of dynamic loading: vertical, lateral, and longitudinal, and it must support these loads safely and economically. This requires an adequate level of strength of the rail, together with a proper support capability of the wheels.

Traditional rail standards, and in particular rail wear standards, are generally strength based, so as to insure that the rail can adequately support this traffic without failure, e.g., fracture under traffic. By combining strength based wear standards with ongoing monitoring of fatigue failures (fatigue standards), railway maintenance officers define a zone of safety for the rail, beyond which the rail must be removed from track.

In addition, rail represents a major cost area in the maintenance of the track structure, representing, for main line freight railroads, as much as 50% of the total variable cost of track maintenance. Thus, the decision as to when and where to replace the rail is an important one, not only from the point of view of safety, but also from the point of view of cost and economics. Leaving rail in track for too long can result in a service failure and the potential for a derailment. Removing a rail prematurely translates into significant costs for the railway. Thus, maintenance officers must maintain a proper balance between safety and cost control. In the case of rail, this is done through the use of cost effective standards for the rail that maintains an adequate margin of safety for the track structure.

To add to the complexity of maintaining these adequate standards, evolving operating conditions and maintenance practices have resulted in significant changes in the way railways determine when rail should be replaced in track. These changes stem directly from changes in maintenance of way practices and materials that have occurred during the past two decades, i.e., better higher strength rail, cleaner steel, improved lubrication and grinding practices, etc., as well as from changes in operating practices, i.e., heavier trains, increased axle loads, higher operating speeds, etc. The net result of these changing practices has been the extension of the service life of the rail, and often an overall reduction in rail maintenance costs over that life (1). We have seen, for example, the decreasing importance of rail joints, and the dramatic extensions of rail life through the use of effective lubrication, grinding, and improved steels (2). While increasing axle loads have resulted in an increased emphasis of fatigue defects, rail wear remains a key replacement criterion for all rail systems, to include freight, passenger, and transit systems. Thus, the importance of maintaining appropriate and adequate rail wear standards likewise remains.

Recently, increased attention has been paid to the wheel/rail dynamic environment of the track structure, with major emphasis placed on the shape of the wheel and the rail (3). This has led to a better understanding of several classes of derailments, to include those wheel climb derailments associated with excessive wear of the rail and/or the wheel (4). This has become of even greater importance in recent years, as several classes of derailments have been associated with these levels of wear. It is the focus of this paper to examine those conditions, and to identify those rail wear criterion and standards, that can reduce the potential for occurrence of these classes of wheel climb derailments.

*President, ZETA-TECH Associates, Inc.

Traditional Rail Wear Standards

Traditional rail standards for the determination of the replacement point for rail in track include wear standards, fatigue (internal) defect standards, surface defect standards, and joint standards. Of these standards, rail wear standards are generally well defined and codified for main line and secondary tracks, with most rail systems (freight, passenger, and transit) having defined wear standard tables in their maintenance of way standards books. This reflects the railways concern about the maintenance of the strength of the rail section, and the corresponding loss of strength due to rail wear.

Rail wear, in turn, is generally divided into two broad categories corresponding to its locations on the rail head. These two categories of rail are:

- Head wear or wear on the top running surface of the rail ("h," see Figure 1).

This wear is most commonly associated with tangent and shallow curves where the wheel/rail contact is exclusively on the head of the rail. Head wear will also occur on curves, particularly severe curves, however, in that environment side or gage face wear will usually dominate.

- Side or gage face wear, i.e., wear on the side of the rail head ("g," see Figure 1).

This wear is caused by contact of the side of the rail head with the flanges of the wheel and is associated with moderate to severe curves. Note that flanging generally takes place on curves 3 degrees and greater, although flanging can occur in shallow curves and even on tangent track (due to hunting).

The corresponding limits or standards for these wear parameters are generally based on the bending strength of the worn rail section (though historically vertical wear has also been defined by the need to avoid wheel flange contact on the top of the joint bars). In order to permit adequate definition of these standards, permissible rail wear limits have been defined in several interrelated ways, as illustrated in Figure 1. These include:

- Allowable vertical head loss (h)
- Allowable side (gage face) head loss (g)
- Linear combinations of head and side wear
- Allowable head area loss (as percent of head area)

Traditionally, North American rail systems have used the first two limits, head and side wear, because of their ease in measurement and relative simplicity of application.

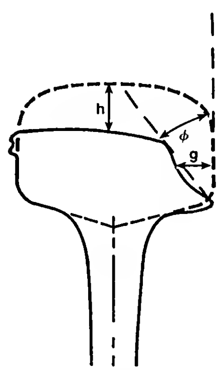

Figure 1. Rail Wear Criterion.

Allowable Vertical Head Loss (h)

Vertical head loss refers to the loss of metal from the top surface of the rail head, as illustrated by parameter "h" in Figure 1. As noted above, vertical head loss occurs on all track, but is a dominant mode of wear on tangents and shallow curves.

Vertical head wear results in a direct loss of rail section strength, since the moment of inertia of the rail section (and thus its bending strength) is related to the rail section height raised to the fourth power. Thus head wear translates directly into a loss of rail bending strength and a corresponding increase in rail bending stresses. When these stresses are greater than the allowable rail stress, then the rail should be replaced for wear (rail wear limit).

Vertical head loss, as defined here, is measured independent of any other parameter, and is taken at the center of the rail head. Vertical head loss is a relatively easy parameter to measure. Both manual and automated methods of measurement are commonly used.

Allowable Side (Gage Face) Head Loss (g)

Gage face wear or side head loss refers to the loss of metal from the inside or gage side of the rail head, as illustrated by parameter "g" in Figure 1. Gage face wear occurs primarily on curved track when flanging of the wheels occurs. It is most pronounced and dominant for curves greater than 3 to 4 degrees. However, it can also be found on tangents and shallow curves, particularly when hunting or truck skewing occurs.

Gage face wear is usually measured 5/8" below the top of the rail head (the gage point of the rail (5, 6)). Thus, its exact location is dependent on the degree of vertical head wear. Gage face wear also results in a loss of rail section strength (vertical) as well as the lateral bending strength of the rail head. When this loss of strength is sufficient to permit the development of stress levels greater than the allowable stress limits (under lateral as well as vertical loading), then the rail should be replaced (wear limit).

Gage face wear is also a relatively easy parameter to measure, with both manual and automated methods of measurement commonly used.

Linear Combinations of Head and Side Wear

In order to account for combined head wear and gage face wear, a linear combination of the two wears (h + g) can be used to establish limits. This allows for the reduction in head metal, and thus rail bending strength, associated with both head and gage face wear. However, it requires the combining of two independent measurements into a "surrogate" measure that, by itself, does not have a physical meaning. It also requires different standards for high and low rails, since the relationships between head and gage wear is significantly different for these two rails.

While this parameter is relatively easy to measure with both manual and automated techniques (since it is simply a linear combination of the above two measurements), this approach has been used only on a limited basis by one or two railroads and no transit systems, due to the difficulty in defining meaningful values.

Allowable Head Area Loss (as Percent of Head Area)

A more accurate method of accounting for combined gage face and head wear is through the calculation of the area loss of metal, and specifically the loss of head area. This value, referred to as head area loss, is usually given in the form of a percentage (%) of the head area that has been worn away.

While this is a more accurate way of determining total wear and the corresponding loss of rail bending strength, this parameter cannot be readily obtained in the field with any standard measuring tool. It requires a detailed rail head profile, such as would be obtained by a profile measurement system (as mounted on an inspection vehicle) and a follow up area analysis. With the increased use of automated optical rail wear measurement systems, this parameter can be more readily obtained and thus has the potential for increased usage in defining rail wear limits. Too date, however, most rail systems standards do not account for this parameter.

The above wear parameters represent the "traditional" parameters used by rail systems to define their limits. As already noted, they are all strength based, derived from the bending strength of the rail (primarily in the vertical orientation, but also in the lateral orientation). Thus, they are all aimed at determining whether the rail section has adequate strength to support the traffic loading, to include dynamic impact loads that have been measured to be three or more times the static wheel load (7). Their intent is, therefore, to insure that the rail does not break under traffic.

Recent research, however, has indicated that wear affects not only the strength of the rail, but also the dynamic interaction between the wheel and the rail. This, in turn, influences not only the wheel/rail loading environment (3), but also the potential for dynamic wheel climb and its associated modes of derailment (4). Thus, it becomes necessary to extend the wheel wear standards beyond these traditional strength based standards, and to address the wheel climb potential associated with the wear of the rail head.

Wheel Climb Derailments and Gage Face Wear Angle

Dynamic wheel climb is a class of derailments most commonly associated with high levels of lateral loads and corresponding high L/V ratios (ratio of lateral wheel/rail force to vertical wheel/rail force). Dynamic wheel climb has been reported for all modes of rail operations to include freight, passenger, and transit operations. Significant research has been directed towards the mechanisms associated with dynamic wheel climb derailments with derailment criterion developed by such researchers as Nadal, Weinstock, and others (8).

Dynamic wheel climb derailments are most commonly associated with sharp curves, where high levels of lateral wheel/rail force are generated. Similarly, wheel climb derailments are found on turnouts, particularly in the curved portion of the turnouts, again where high levels of lateral wheel rail forces have been developed. One survey of turnout related derailments in transit systems (to include both main line and yard derailments) has found approximately 40 reported wheel climb related derailments, which corresponds to more than 40% of all turnout related derailments (track caused) reported in that study.

Among those factors that have been reported to contribute to this class of wheel climb derailments is the angle of the gage face of the rail, usually the outside or high rail of the curve. This angle is often found on rails subject to gage face wear, i.e., outside or high rails, where this gage face wear can result in the development of an angle ϕ between the gage face and the vertical (see Figure 1). As this angle increases, the potential for a wheel to climb the gage face of the rail increases. This wheel climb will occur when the net "upward" component of the lateral (L) and vertical (V) wheel/rail forces, parallel to the rail gage face, is greater than the resistance to that force due to the normal force component N (see Figure 2) and the corresponding coefficient of friction f, i.e., N • f (9, 10). Thus wheel climb can occur when the following relationship occurs (see Figure 3):

$$L/V < \tan(\beta - f') \qquad (1)$$

where:

L = Lateral wheel/rail force
V = Vertical wheel/rail force
$\beta = 90 - \phi$ (see Figure 3)
$f' = \tan^{-1}(f)$

and

f = coefficient of friction

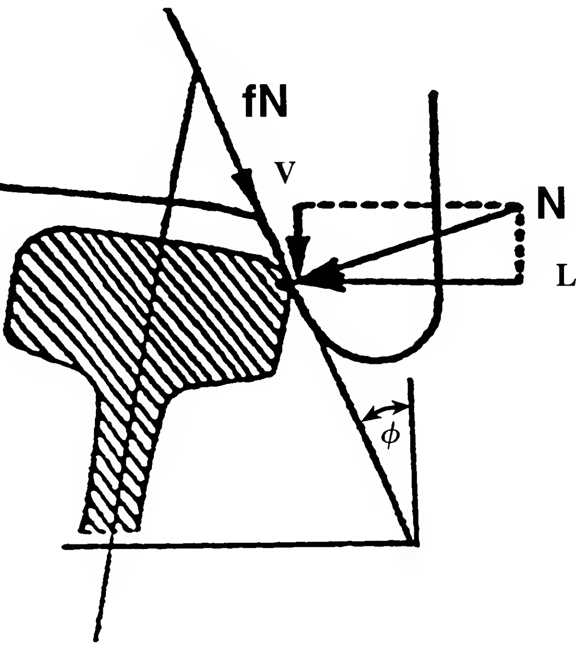

Figure 2. Wheel Climb Potential.

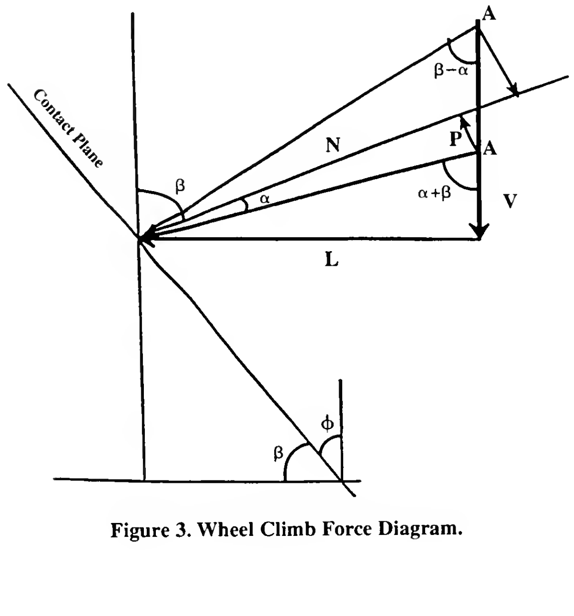

Figure 3. Wheel Climb Force Diagram.

Thus, based on the above equation, as the gage face wear angle ϕ increases, the corresponding level of L/V ratio required for wheel climb decreases, thus increasing the risk of wheel climb derailments. In order to reduce this risk, some rail properties have introduced a standard for the maximum angle of side wear of the rail. This criterion has been introduced recently on several United States rail systems, in some cases shortly after investigation of a wheel climb derailment found that this wear angle was a contributing factor. This was the case recently on BART (San Francisco) and PATCO (New Jersey). This criterion has also been extensively used in Europe and Asia, where rail gage face angle limits have been used on such rail systems as British Rail (11), Deutsche Bundesbahn (12), Indian Railways (13), and the Netherland State railways (9). This criterion has recently been adopted by a number of U.S. rail systems, including Amtrak, and several U.S. transit systems, as a supplemental safety standard to their rail wear standards.

Until recently, measurement of this gage face angle had been difficult to accomplish, usually requiring special gages which have been developed by individual properties. However, with the growing use of optical rail wear measurement systems which define a complete rail head profile accurately and reliably (14), it is possible to calculate this rail gage face angle value directly from the measured rail head profile. This is, in fact, the approach that has been introduced by Amtrak, as part of its ongoing rail wear inspection program using a contracted optical wear measurement system. By defining an algorithm for the calculation of gage face wear angles, as part of the analysis of the rail profile data, and comparing these angles to preset thresholds, exception reports are obtained from those locations where the rail gage face angle values exceed these thresholds.

It should be noted that the actual values used for these standards vary, based on the expected levels of L/V ratio for the operating equipment and operating conditions. Thus, limits for wheel climb that have been used by different properties vary from between 26 degrees and 32 degrees, depending on the level of expected L/V ratios and the desired margin of safeties. This will be discussed in further detail in the next sections.

Wheel Climb Sensitivities

As can be observed in Figure 3, the wheel/rail forces normal and parallel to the gage face (and thus associated with this risk of wheel climb) are functions of the lateral (L) and vertical (V) force values and their corresponding ratio (the L/V ratio). They are also sensitive to the angle of gage face wear ϕ and the coefficient of friction (f) (see equation 1). Since the potential for wheel climb is directly related to the L/V ratio, it is possible to define those combinations of L/V ratio, gage face wear angle (ϕ), and coefficient of friction (f), which introduce an unacceptable level of risk for wheel climb.

Figures 4 through 9 present such a sensitivity analysis, which shows the calculated L/V ratios associated with wheel climb as a function of different gage face angles and coefficients of frictions. As can be seen from Figures 4 and 5, increased levels of friction, corresponding to dry or unlubricated rail, reduce the L/V ratio required for wheel climb at a given gage face angle, thus increasing the potential for wheel climb by bringing the level of loading required down to a magnitude where this level of loading has in fact been measured. While normal lubricated rail has a range of friction of the order of 0.10 to 0.35, depending on the level of lubrication, dry or unlubricated rail can have a coefficient of friction of the order of 0.45 or higher (with values as high as 0.6) (15). Thus, effective lubrication not only has the ability of reducing the level of lateral loading (16), but also increases the level of loading needed for wheel climb to occur, even at high gage face wear angles. Conversely, dry or unlubricated rail can directly increase the risk of wheel climb derailments. In fact, in several of the derailments noted above (i.e., PATCO, BART), the gage face of the rail was very dry at the time of the derailment, thus contributing to the derailment itself.

To illustrate this behavior, for a well lubricated condition, with a coefficient of friction of 0.2 and a gage face wear angle of 28 degrees, the level of L/V required for wheel climb is 1.20, a very high level rarely seen in service. However, for a very dry condition, corresponding to a coefficient of

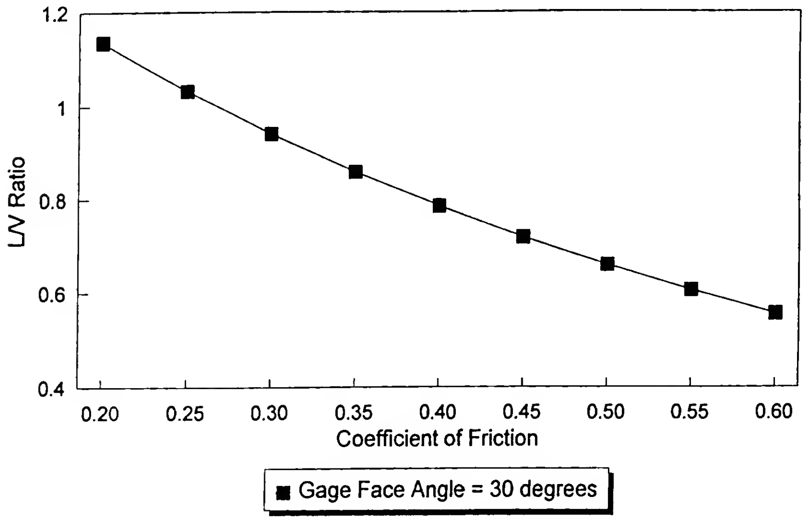

Figure 4. Wheel Climb Criterion. Gage Face Wear Angle Limits.

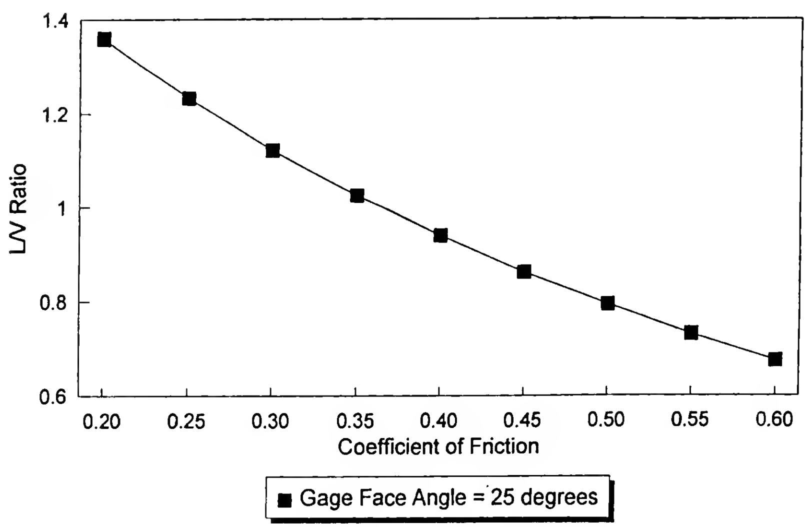

Figure 5. Wheel Climb Criterion. Gage Face Wear Angle Limits.

friction of 0.5, the same amount of gage face wear angle results in a L/V level required for wheel climb of 0.70. This is a level that has been measured in the field on a regular basis, and is, in fact, below the traditional Nadal wheel climb threshold of 0.8.

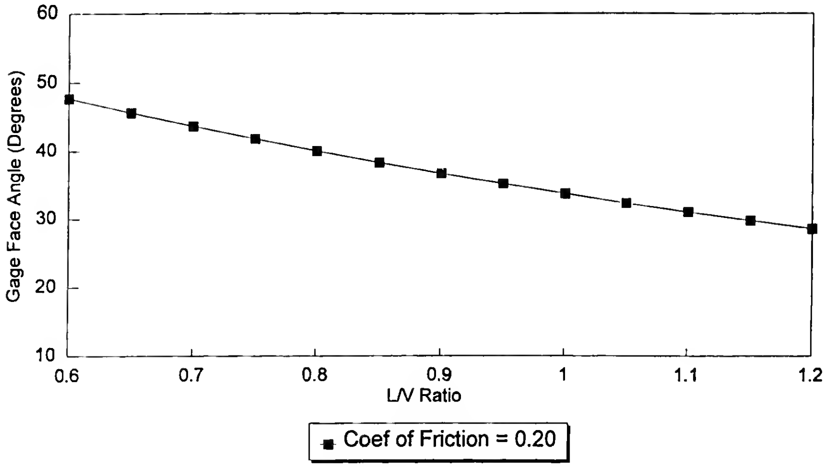

Figure 6. Wheel Climb Criterion. Gage Face Wear Angle Limits.

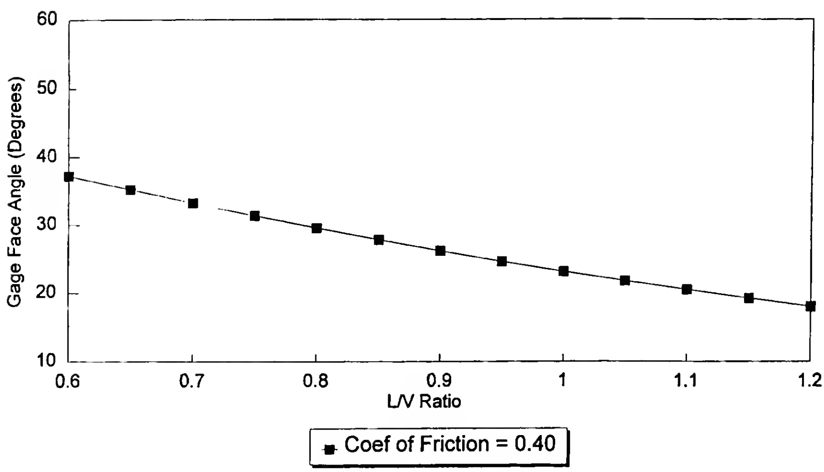

Figure 7. Wheel Climb Criterion. Gage Face Wear Angle Limits.

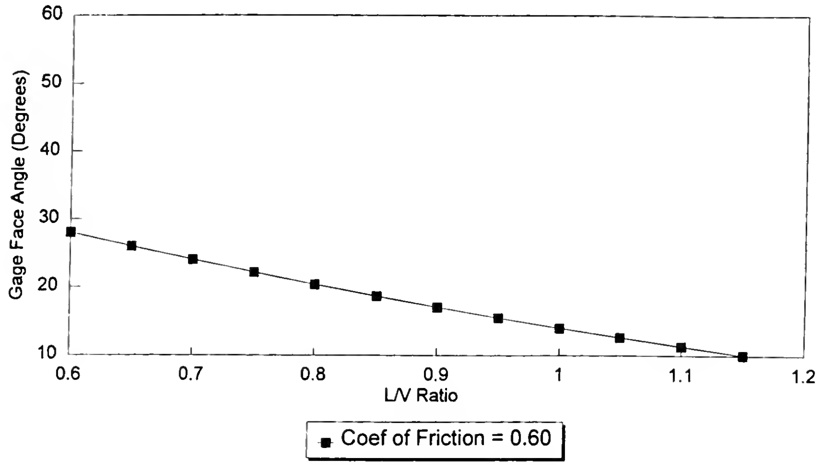

Figure 8. Wheel Climb Criterion. Gage Face Wear Angel Limits.

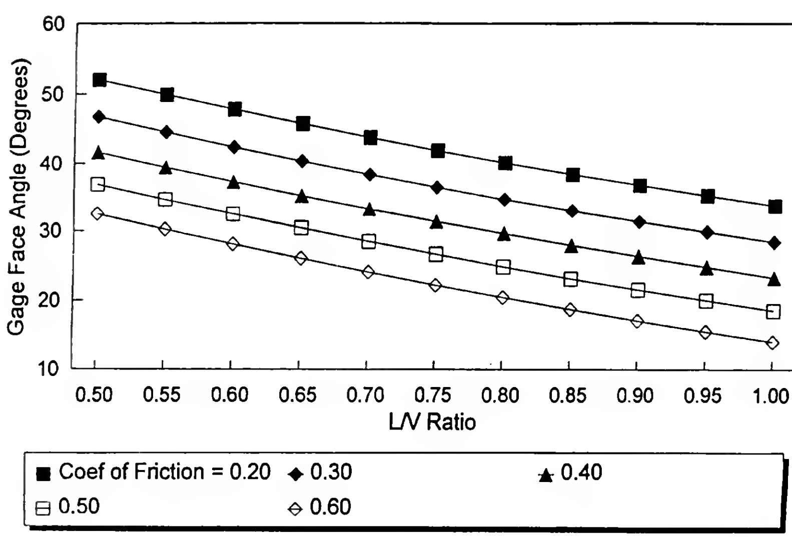

Figure 9. Wheel Climb Criterion. Gage Face Wear Angle Limits.

Likewise, increasing the allowable gage face angle, i.e., allowing a steeper "slope" at the gage face while holding coefficient of friction constant (see Figures 6 through 9), will result in a reduction in the L/V ratio required for wheel climb and consequently increasing the potential derailment risk. Thus, as can be seen in Figure 7, for a coefficient of friction of 0.4, a gage face angle of 18 degrees requires an L/V ratio of 1.20 for wheel climb, an extremely high level which is almost never achieved, while a gage face angle of 32 degrees requires an L/V ratio of 0.7. As noted above, this is a value which has been measured in the field for a range of vehicle types and operating conditions.

It can thus be seen that by setting a maximum limit to the gage face wear angle, it is possible to reduce the risk of wheel climb by forcing the wheel climb requirements to a level of loading (L/V ratio) that will not be encountered in the field. Alternately, if the maximum L/V ratio that occurs on track is known (through field measurements, tests, analytical modeling, etc.), together with an appropriate coefficient of friction, then it is possible to calculate the maximum allowable gage face angle above which wheel climb may occur. This would thus be the limit for maximum allowable gage face wear angle.

Recent Experience and Practice

As noted above, gage face wear angle limits can be used to control the risk of this class of wheel climb derailments. These limits can be defined most effectively, if the actual levels of lateral (L) and vertical (V) loadings are known, thus defining the range of L/V values for the system. This approach was used by such properties as Amtrak and PATCO in setting their gage face wear angle standards. In both cases, test data were available which defined the range of loadings and L/V ratios expected in service. Using these L/V ratio limits, and knowing the lubrication practices, and thus, expected coefficients of friction, it was possible to define maximum allowable gage face wear angles so as to avoid wheel climb.

For example, in the case of PATCO, maximum measured lateral load values (L) were available from previous field tests. Using the worst case measured lateral load levels, combined with the lightest vertical loads (usually unloaded equipment), it was determined that the L/V ratio for this equipment was always less than 0.84 for unguarded curves (no guard rail). Defining a light to moderate level of lubrication of (0.4) results in a gage face wear angle limit of 28 degrees. Noting that drier rail (higher coefficient of friction) will reduce this angle limit (see Figure 7), a gage face limit standard of 26 degrees was defined.

Amtrak likewise made use of available loading and L/V data to define a gage wear angle limit of 30 degrees (based on a defined set of operating conditions, corresponding to that level of loading). Amtrak furthermore made use of its contract rail profile measuring activity to inspect for this wear angle and is currently experimenting with the threshold limit for use in the generation of "exception" reports to define locations with excessive gauge face wear angle values. Currently a "maintenance" limit of 27 degrees is being used to define exception locations.

Other U.S. properties, to include BART (San Francisco), PAT (Pittsburgh), and SEPTA (Philadelphia) have defined gage face wear limit values between 26 degrees and 32 degrees, based on the anticipated (or measured) levels of loading, and standards for lubrication.

While most freight systems in North America have not adopted gage face wear standards, as of yet, their European counterparts have. In Europe, where this parameter is frequently used, standards likewise range from 26 degrees to 32 degrees.

Thus, for example, British Rail uses a gage face wear angle standard of 26 degrees (11), defined as illustrated in Figure 10 (note this angle corresponds to angle ϕ in Figure 1). As noted in reference 11, this limit is imposed to avoid derailments, i.e., to prevent the class of wheel climb derailments already noted above.

Other European and international railways likewise use a gage face wear angle limit or alternately use a more restrictive limit to measure gage face wear, in lieu of the angle limit, in order to

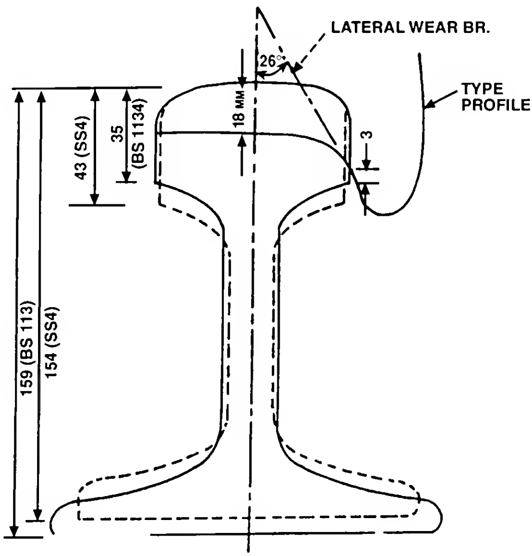

Figure 10. British Rail Standards.

restrict wear and prevent excessive gage face wear angle values. Thus, for example, on the NS (Netherlands Railways), the limit for lateral (gage face) wear is 8 mm (.32 inches) without any gage face angle limit, or it is extended to 12 mm (0.5 inches) if a gage face angle limit of 32 degrees is imposed (9). Likewise Indian Railways define limits of lateral wear and angle of wear to avoid the "risk of wheel mounting the rail caused derailments" (13).

Finally, the issue of measurement of this gage face wear angle value should be addressed. While several rail systems have developed hand gauges which can be used to measure gage face wear angle (e.g., BART, PATCO), either as a direct measurement or as a go/no go gauge, current rail profile measurement technology now allows for the measurement of the complete rail section. This approach, most commonly using optical techniques which have been used by numerous rail systems both in North America and overseas, readily lends itself to the calculation of this gage face angle. This is the approach currently used by Amtrak, where by defining a "zone" on the gage face of the rail, the actual wear angle is calculated and compared to the defined standard (be it a safety or maintenance standard). Exception locations can then be identified, and if desired, an ongoing record of condition maintained. This approach is expected to be used more and more in the future, since it can be readily incorporated into current rail wear measurement practices with the incorporation of an additional calculated parameter.

Summary

Traditional rail standards, and in particular rail wear standards, are generally strength based, so as to insure that the rail can adequately support traffic without failure. In this approach, a zone of safety is defined beyond which the rail must be removed from track. In addition, rail represents a major cost area in the maintenance of the track structure. Thus, the decision as to when and where to replace the rail is an important one, not only from the point of view of safety, but also from the point of view of cost and economics. As axle loads increase and operating conditions become more severe, the importance of maintaining safe and appropriate rail wear standards becomes of even greater importance to cost conscious railroads.

As railroads' understanding of the wheel/rail dynamic environment of the track structure increases, a better understanding of several classes of derailments, to include those wheel climb derailments associated with excessive wear of the rail, has emerged. These dynamic wheel climb derailments are a class of derailments most commonly associated with high levels of lateral loads and corresponding high L/V ratios. Dynamic wheel climb has been reported for all modes of rail operations to include freight, passenger, and transit operations.

Among those factors that have been reported to contribute to this class of wheel climb derailments is the angle of the gage face of the rail, usually the outside or high rail of the curve or in a turnout. As this angle increases, the potential for a wheel to climb the gage face of the rail increases. This wheel climb will occur when the net "upward" component of the lateral and vertical wheel/rail forces, parallel to the rail gage face, is greater than the resistance to that force.

By defining standards or limits for these gage face wear angles, the risk of this class of wheel climb derailments can be controlled. This approach has been used by numerous rail systems both in North America and overseas to reduce derailment risk for this class of derailments. In the case of several U.S. properties, test data which defined the range of loadings and L/V ratios expected in service was used, together with lubrication condition (which defined the coefficient of friction between the wheel and the rail) to define maximum allowable gage face wear angles so as to avoid wheel climb.

In the U.S., several properties have defined gage face wear limit values between 26 degrees and 32 degrees, based on the anticipated (or measured) levels of loading and standards for lubrication. Likewise, in Europe, where this parameter is frequently used, standards similarly range from 26 degrees to 32 degrees.

In order to measure these angles, several rail systems have developed hand gauges which can be used to measure gage face wear angle, either as a direct measurement or as a go/no go gauge. However, current optical rail profile measurement technology allows for the measurement of the complete rail section, and thus readily lends itself to the calculation of this gage face angle. This is the approach currently used by at least one U.S. property to calculate the rail wear angle and compare it to a defined standard (either safety or maintenance standard). This approach is expected to be used more and more in the future, since it is readily incorporated into the current rail wear measurement practices.

Acknowledgment

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SEPTA RAILWORKS—TAKING LINE OUT OF SERVICE

By: C. L. Rood*

On behalf of the Southeastern Pennsylvania Transportation Authority, more commonly referred to as SEPTA, I would like to thank the AREA for expressing an interest in SEPTA and the SEPTA RailWorks project.

My presentation covers some of the planning, service detour scenario, and construction aspects for one of SEPTA's larger capital improvement programs, called RailWorks, which was substantially completed in late 1993—completed while taking the mainline out of service. The core element of this $290 million dollar project was the replacement and/or rehabilitation of 25 bridges on a 4 mile section of SEPTA's Regional Rail Division, a division which provides heavy rail passenger services for over 85,000 daily riders in and around the city of Philadelphia.

But before discussing the project, let me talk briefly about SEPTA. SEPTA is one of the largest public transportation agencies in the nation. Its service area includes the five (5) southeastern counties of Pennsylvania which is an area with a population of over 3.7 million. It has a combined operating and capital budget for Fiscal year 1996 in excess of *$865 million*.

SEPTA operations include:

- A fleet of over 1,400 buses operating on 110 routes through Philadelphia and its suburban counties;
- 164 track miles of elevated, subway, and surface light rail transit; and
- 485 track miles of heavy rail passenger service, stopping at 160 stations.

The heavy rail passenger service, or Regional Rail Division, is made up of 13 electrified branch lines of mixed single and double track territory, which feed into an eight-mile, 4-track *central* trunk line. This main trunk line runs underground through the Philadelphia central business district via the Center City tunnel, runs northwest above grade on a series of bridges and retained earth viaducts to Wayne Junction in North Philadelphia, and west to Amtrak's 30th Street Station in West Philadelphia. Except for the Center City tunnel section which connected the Old Reading with the Old Pennsy, and the Line to the Philadelphia Airport, both completed in the early to mid-1980's, most of the Regional Rail service operates over bridges and other physical plant originally built *prior* to 1910 by either the Pennsylvania or Reading Railroad.

Starting in the early 1980's when SEPTA took over ownership and direct operating responsibility for the passenger *rail* network, we began to make large capital investments in upgrading and modernizing the physical plant using federal funds made available. Major improvements and upgrades have been and continue to be made in the areas of track, signals, electrification, structures and equipment.

Since 1984, SEPTA's capital bridge program, for example, has replaced or rehabilitated more than 25% of the *257* bridges on the Regional Rail system which needed some level of attention. Many were typical bridge replacements/rehabilitation done in a manner dictated by the service disruption allowed.

The enormous scale of the bridge reconstruction project made RailWorks considerably different from a typical mainline railroad track or bridge project. We couldn't just use a service disruption scenario utilized for bridge replacements in the past. We opted for an extended complete shutdown. The scope of the physical plant improvements incorporated into the RailWorks project was quite extensive with planning alone beginning in mid-1985 some 6 years before construction. The project included bridge work to catenary replacement and everything in between.

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The project is located on the north end of the 4-track central trunk line I discussed earlier. Work started just north of the new Center City Tunnel Construction, which was completed in early 1983, and which as previously mentioned combined 2 systems—the Pennsy and the Reading, and covered 4 miles to Wayne Junction where the main trunk converges into a 2-track operation. Because of the way our northern and southern branch lines are paired together, all *400+* of our daily regional rail trains operate over this section of the system.

Early in the start-up of our capital bridge program, the majority of the 25 bridges along this 4-mile section were identified for replacement due to their state of deterioration and key location on our central trunk line. Deciding how to replace these 25 bridges quickly, safely, in a quality fashion, at a cost we could afford, with the least impact to the community, local business and passengers, and while operating passenger service on a reliable schedule was the subject of considerable study and analysis. We knew it wasn't business as usual. A planning process as previously mentioned took close to 6 years.

Here's what we were faced with. The 4 mile section which RailWorks encompasses runs through a section of North Philadelphia, beginning in Center City and running northward, through an area which is mainly densely residential and light commercial. The bridges in this area had all been built between 1906 and 1911, as part of a grade separation project on-going at that time, and they had little if no real maintenance for the past 40 years. Their waterproofing and drainage systems had failed causing deterioration even to the patchplates installed. Most had non-functional bearings and several had deterioration of many of their bridge members.

For the most part, the bridges were 3 span riveted steel construction with the main span over the street and the approach spans over the adjacent sidewalks with the bridge columns on the curb line. There also was a two-span open deck bridge spanning over one of Conrail's active lines. Of the three span bridges, some were through girders with a built up trough floor system, and some were deck girder design.

There were also two un-reinforced concrete arches, which early in the design were slated to be rehabbed, but the results of our preliminary concrete investigation indicated that the arches had reactive concrete and, therefore, the arches were slated for replacement.

The bridges weren't the only major bad portion of the fixed plant in this 4 mile section. The overhead catenary system was also in need of replacement virtually having had little maintenance since its installation in 1935. Portions of the track structure, including two major interlockings, were also in need of extensive work. Combining these needs with the bridges and other needs within this corridor, the project evolved into a *Systems Infrastructure Improvements Project.*

The major work items included:

- Replacement of 21 bridges and repair of another 4 with all bridges being four tracks wide;
- Replacement of 8 miles of track;
- Renewal of 16 miles of overhead catenary;
- Replacement of 2 Interlockings;
- Installation of new signal and power control cables;
- Construction of a new Temple Station, and
- New right-of-way fencing, retaining wall work, catenary structure rehabilitation, duct bank installation, and the rebuilding of the existing station at North Broad.

Performing the bridge replacements, and other civil and electrical work eventually added to the project, under service would have exposed our service to daily system-wide delays over at least a 5-year construction period, and construction costs would have more than doubled.

We ultimately decided to suspend service over this 4 mile section of the system and package all the construction into two consecutive construction windows of 6 and 4 months respectively. The decision to suspend service did not come easy. In early 1986, we looked at five different scenarios.

- *Scenario A*: A full service scenario. All four tracks are in service on weekdays. Construction activity would be limited to weeknights and weekends.
- *Scenario B*: This scenario limits operations to three tracks; one track would be continuously available to the Contractor.
- *Scenario C*: This scenario limits operations to two tracks; two tracks would be continuously available to the Contractor.
- *Scenario D*: This scenario calls for the complete shut-down of operations; all four tracks would be continuously available to the Contractor.
- *Scenario E*: This scenario was a combination of scenarios C and D.

For each, we did a detail analysis of its:

- Operational Feasibility
- Engineering Feasibility
- Estimated Cost and Duration
- Estimated Revenue Loss
- Required Physical Facility Revisions

We had at one time looked at three consecutive construction windows of 3.5, 4, and 4 months respectively. We rated each scenario giving an A grade to the best and a D grade for the worse, and due to the interest of both time and money and least service impact, we opted to go with the 2 construction window scenario or Scenario D—The Service Detour.

We made this tough decision mainly because we were able to come up with a reliable alternate service plan. We utilized the summer months due first to ideal construction weather, and secondly, we have less ridership over the summer months due mainly to school closures and vacations.

The alternative service plan developed and ultimately utilized consisted of splitting the Regional Rail service into two halves, south and north of the construction zone. Through a combination of alternate busing and the running of two stubbed end terminal operations on the Regional Rail a shutdown scenario became possible. South of the project, two existing center city stations (Suburban & Market East) were used for the south side stub operation. On the north side, our subway line runs close to our Regional Rail line, so we built a new interchange station at Fern Rock between our Regional Rail service and our Broad Street Subway line. This provided a convenient short walking transfer point for rail riders of the Reading side of the R2, R3, and R5 lines to switch to the subway to continue their trips to Center City or beyond. The Reading side of the R6 and R7 Lines and the R8 Fox Chase Line were completely suspended and substitute bus service provided.

The new interchange station at Fern Rock was just one of several support projects which were completed as part of our alternate service plans.

To accomplish all of the work, it was broken up into manageable pieces, with specific construction contracts, phased over the two construction periods mentioned earlier.

There were:

- 6 General Construction Contracts;
- 2 Electrical Construction Contracts; and
- 1 small Mechanical Construction Contract (Temple Station).

The Contracts values ranged from $.25 million to $27 million.

The phasing and packaging was basically dictated by street detouring capability, attempting to minimize impact on the neighboring communities and access and staging requirements necessary by each of the contractors. Many access points and staging areas were obtained in the planning stages, well prior to actual construction, via lease agreements with the necessary neighbors.

We had 3 of the 9 total contracts working over both construction periods, and the other 6 working either the first or second period. In the first window, we had electrical, 1 mechanical and 4 general contractors, and in the second window, we had 1 electrical, 1 mechanical and 3 general contractors.

The *new* bridges, with the exception of the two arches, were basically updated versions of the original bridges, but in a modern design. The bridges were either through girder or deck girder design with new deck floor systems of either steel plated decks or concrete encased floor beams and stringers. Where possible, we attempted to eliminate the curb line columns, but were successful in doing so at only 3 locations.

We had an envelope of space to fit the bridges into. We had to maintain or improve the under-bridge roadway clearances and we were restricted with our overhead catenary height and/or our track line and grade.

The abutments were all reused, although some reactive concrete problems existed; however, all were massive gravity abutments and, therefore, highly stable. The replacement of the bridge seats and backwalls was done at most of the bridge locations. The typical scenario for the steel bridge replacements was that, after the power systems were removed, the track system, including the deck and waterproofing system, was removed. The bridges were either torch cut out or torn out. Once the steel was removed, the old backwalls and bridge seats were removed. The new construction then began—new backwalls and bridge seats were installed and the new steel was erected. The majority of the new bridges were erected in place but two were done as *roll-ins*. This was primarily the contractor's decision, since he felt he didn't have sufficient time in the window to complete all of his bridges unless he proceeded that way. And, he made all the necessary arrangements with the adjacent neighbors. Once the steel was in place, the concrete decks were poured. After the deck installation, the waterproofing and drainage systems were done and the track system, consisting of new ballast, new wood ties, and new CWR on pandrol plates, was completed.

The replacement of the concrete arches was a little unique since incorporated in their replacement were the entrances into the new Temple Station. Once the power systems and track systems were removed, demolition on the arches began. The arches, along with their north abutment, were removed and excavation for the new north abutment began. We dug a big hole and then began to fill it—first with lean concrete, which was resting on rock, and then with the rest of the abutment. After the abutments and piers were complete, steel was erected. The deck girders were completed, along with portions of the high level platform footings. After the concrete deck and waterproofing and drainage systems were installed, the track system was renewed.

The station, with the exceptions of its footings, was completed in the second window, with some work continuing through the time between the windows.

The four bridges that were rehabbed were basically *all* done in a similar manner. Our largest rehab was a separate general construction contract in itself. The rehab was of a 3,400 foot long steel viaduct approximately 60 feet wide, which runs directly over 9th Street. The deck was stripped of the track structure and existing waterproofing system. The existing concrete deck overlay was removed and replaced with a fiberized concrete overlay. Because of access difficulty, the contractor did those tasks (overlay removal and installation) in half lengthwise. After the deck was completed, along with new parapets, the new waterproofing, drainage, and track systems were installed. The underside work included rivet replacement, deteriorated structure member replacement, bearing replacement, and cleaning and painting of the structure.

Other work in the project was the renewal of two interlockings. At one interlocking, situated in a cut section, everything was completely gutted. Installed then was a new drainage system, new sub-base, new ballast, rail, ties, new special track work, new duct bank, new signal and power cables, and new air operated switch machines.

At the other interlocking, all the special track work and electric switch machinery were replaced. That interlocking, located at the extreme north end of the project, was done without a complete shut-down, since we had to maintain access into both ends of our Roberts Yard car shop. The work had to be done under service with scheduled shutdowns within the construction window.

Also done as part of this project was the replacement of approximately 16 miles of overhead catenary—a three (3) wire inclined system. Rehabilitation consisted of steel repairs; cleaning and painting of all the catenary structures, not renewed for the station alignments; running of new signal cables; wall work; right-of-way fencing; and an underground duct system, to house the power, signal, and communication cables.

With the limited amount of time, much of the work had to be done around the clock 6 to 7 days per week.

It was indeed a tight construction schedule!

The time spent in pre-construction planning for this job was enormous, with over 6 years of planning and design for 10 months of actual construction. The construction contracts were bid and subsequently awarded up to 2 years prior to construction. This was mainly done to allow the contractors themselves sufficient time to *plan* their respective work and also to procure their material, especially since much of the material required a long lead time. To stress the importance of this project to the contractors, many contractual additives were placed in the contracts. In addition to major milestone dates previously mentioned, and potential liquidated damages; to ensure material deliveries, a contract requirement was that all the major materials had to be on site and inspected at least 70 days prior to the respective window, with the bridges being required to be shop fit and inspected *prior* to shipment. Also, the special trackwork was required to be shop fit and inspected prior to shipment.

Completing a project of this size and complexity in the extremely tight time frame of two construction windows totalling only 10 months, required a tremendous amount of planning, but it also required exceptional *Team* effort. A team consisted not only of SEPTA employees, but also of contractors, consultants, fabricators, and suppliers. To borrow a phrase from our construction manager, "WE HAD CREATED A RECIPE FOR DISASTER." What he was referring to was the large number of organizations involved with the project along with other constraints.

But through a dedicated team effort, we *accomplished our Goal.*

And as seldom heard of in the Railroad industry, we shut our rail mainline down for construction purposes not just once, but twice and not just for a day or two, but for a six- then a four-month duration. As mentioned, it wasn't an easy decision to make. We had good alternative service, a good Public Relations campaign, and the drive to save both time and monies, while minimizing impacts to our customers.

How did our decision impact ridership? We lost riders mainly for the duration of the construction shutdowns, the months between and for a few months after the second shutdown.

We had anticipated that approximately 19% of the former Reading side passengers would leave the SEPTA system during RailWorks. The average Phase 1 retention rate realized on these lines was 72%, a loss of *28%*.

Ridership loss was again projected prior to Phase 2 construction (in between the windows), at about 78% of Pre-Phase 2 levels. Actual average retention during this period was about 85%, a loss of only *15%* and not the anticipated *22%*.

Ridership counts for the week which ended November 6, 1993, 2 months after the second shutdown ended, indicated continued improvement when compared with the periods before Phase 3 construction (+3.5%) and a year ago, immediately following Phase 1 construction (+9.8%). The decline since March, 1992 (pre-RailWorks) had lessened to only a *–7.2%* drop.

However, factors other than RailWorks were contributing to the remaining decline from pre-RailWorks levels, such as the sluggish regional economy, competition with automobiles as drivers experienced improved highway access to Center City Philadelphia and low gasoline prices, service reductions, and a continuing decentralization of the urban job market. Regional rail lines not affected by RailWorks also experienced a decline of approximately 9% between March 1992 and September 1993.

But, through a concentrated effort, ridership on the *affected* lines came back. Within 4 to 5 months after the second phase of RailWorks, ridership was back to pre-RailWorks levels. Much quicker than anticipated, and it has been rising steadily since.

Were the shutdowns the right decision? For us and this project, I think it was.

FALL PROTECTION AND SAFETY CLIMBING DEVICES FOR BRIDGE INSPECTION

By: R. Ross*

Introduction

The bridge inspector is entrusted with great responsibility. He is responsible for the safety of the public as well as that of his fellow employees. His duties help ensure the protection of the environment and the safe keeping of some of his corporation's most valuable assets. In order to provide this protection, he must be able to perform his job quickly and efficiently and yet maintain a high level of quality. He must do all of this without neglecting his own safety. Even though he will often be working at dizzying heights, this does not mean he must place his life on the line.

The average railway bridge inspector working in North America today inspects roughly 500 bridges a year, and some of these more than once a year. This statistic underscores the primary difference between the inspector and the bridge worker—a critical need for mobility. While the bridge worker will often be stationed in the same work location for months at a time, the inspector must be constantly moving from one bridge to the next. On the structure itself, the bridge worker frequently works within limited confines for prolonged periods. The inspector must access every square inch of that structure without lingering in any one location for too long. In recognizing this difference, it should be readily apparent that equipment and techniques used for access and protection will differ too.

It was the mission of this Special American Railway Bridge and Building Association (i.e., B&B) committee to collect and present alternatives in fall protection for the safe, thorough, and efficient inspection of railway bridges. A survey was conducted of North American railroads requesting information on the types of fall protection in use and other pertinent information applicable to bridge inspection. The detailed results of this survey can be found in Appendix A at the end of this report.

Also included in this report is a glossary of rock climbing terms which can be found in Appendix B. Unless you climb or belong to an emergency rescue squad that practices high angle rescue techniques, you will probably be unfamiliar with many of the terms used in this report. Rock climbing techniques have recently been adapted and put into use for bridge inspection on some railroads and also in some governmental agencies responsible for inspecting bridges. Canadian National, New Jersey Transit, and Conrail reported using climbing techniques in their bridge inspections. On the highway side, CALTRANS has just issued a Code of Safe Practice for climbing techniques in bridge inspection. They recently applied these techniques in assisting the Army Corps of Engineers and the Bureau of Reclamation in an emergency inspection of a damaged floodgate where conventional means of access could not be used in the time frame required. The Army Corps of Engineers is currently in the process of working up their own policy. Kentucky DOT has been using rock climbing techniques in bridge inspection for over five years and Ohio DOT has recently begun training its inspectors. The industry appears to be leaning in the direction of accepting climbing techniques as a viable option for bridge inspection. Since many railroads are unfamiliar with these techniques, special emphasis has been placed on climbing in this report to introduce the techniques as an alternative to methods already in use.

FRA Policy And Regulations

49 CFR Part 213—Policy on the Safety of Railroad Bridges: An interim statement of policy, issued April 1995, established guidelines for railroads to use in bridge management. The guidelines are advisory, not regulatory, in nature. Guidelines six through eleven address bridge inspection specifically. In regard to inspection frequency, the policy states:

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6. (b) The prevailing practice throughout the railroad industry is to inspect railroad bridges at least annually. Inspections at more frequent intervals may be indicated by the nature or condition of a structure or intensive traffic levels.

49 CFR Part 214—Bridge Worker Safety Rules: This regulation, issued in 1992 and revised in 1994, governs anyone working on a railway bridge. A height threshold of 12 feet was established at and above which fall protection becomes mandatory. Other detailed requirements for fall protection systems and equipment are detailed and can be referenced in the regulation. Although the original rule did not differentiate between classes of work, the revision dated June 14, 1994 gives the bridge inspector a great deal of latitude in the use of fall protection and reads as follows:

214.103(b)(2) This section shall not apply to employees engaged in inspection of railroad bridges conducted in full compliance with the following conditions:

(i) the railroad or railroad contractor has a written program in place that requires training in, adherence to, and use of safe procedures associated with climbing techniques and procedures to be used;

(ii) the employee to whom this exception applies has been trained and qualified according to that program to perform bridge inspections under the provisions of that program, and has accepted the designation;

(iii) the employee to whom this exception applies is familiar with the appropriate climbing techniques associated with all bridge structures the employee is responsible for inspecting;

(iv) the employee to whom this exception applies is engaged solely in moving on or about the bridge or observing, measuring, and recording the dimensions and condition of the bridge and its components; and

(v) the employee to whom this section applies is provided all equipment necessary to meet the needs of safety, including any specialized alternative systems required.

This revision is important in that it allows the inspector to use his experience and judgment in deciding when and when not to use fall protection. It also places a responsibility on the railroad to ensure that the inspector has the proper equipment, skills, knowledge, and training to properly protect himself and enable him to make sound decisions when selecting a technique or piece of equipment for a particular application.

Access and Fall Protection Alternatives

There are several alternatives available to the bridge inspector if he is properly equipped and trained. Some of these alternatives can be used in combination. All have their advantages and disadvantages. The following criteria have been developed for evaluating and comparing methods:

- Safety
  What level of fall protection does the method offer? Does the method prevent falls or arrest them? Are there opportunities for other injuries to occur such as strains and sprains? Is bulky equipment used which requires multiple setups? What level of fitness is required to perform techniques?

- Thoroughness
  Does the method allow the inspector to get within arms length of the structural member being inspected? Is he able to inspect the entire bridge using this method? Will this method help the inspector meet his responsibility of inspecting all bridges on his territory?

- Efficiency
  How fast is the method? Does the method require track usage? Does the method require interruption of automobile or water traffic? Is extensive training required? How much does the method cost?

Keeping these criteria in mind, here is a breakdown and evaluation of some alternatives available:

Inspection Vehicles

These include underbridge units, aerial lift units, and crane supported hanging baskets. The major advantage of these units is that they are occasionally the only *reasonable* means of access to particular areas of a structure. Also on the plus side, they don't require much from the inspector in terms of physical strength. There is a major downside, however. The railmounted units require track usage. Because of increases in traffic volume on one hand and railroad downsizing on the other, it is becoming increasingly more difficult to compete with rail traffic for inspection time. The highway units also present difficulties requiring lane closures and highway traffic disruption. These units are also very costly to purchase or rent. Most of the railroads surveyed only own one underbridge unit, which must be shared and used over large territories.

Scaffolding and Staging

These access methods make for great work platforms but simply take too much time to set up, move and tear down. They are well suited to the bridge worker but rarely fit the needs of the inspector.

Ladder

This is an indispensable tool. On small structures where there are few setups required, it is invaluable for accessing bearing areas and floor system connections. On larger structures however, limited reach can become a problem and just lugging the thing from one setup to the next can introduce the potential for a back injury.

Nets

This method of fall arrest is very expensive and time consuming to set up and tear down. The erection of such systems is usually beyond the scope of work conducted by railway personnel and is handled by private contractors. This type of protection is normally used to protect workers on a large project that justifies the expense and is not well suited for use in bridge inspection. A variation on this idea has been used to access floor system connections using the net as a platform and using an independent system to anchor the bridge inspector. This method has proven effective where access is difficult or impossible using other means.

Lifelines

Horizontal lifelines are currently in common use by bridge and track workers on bridges. They provide a continuous means of protection, but often limit the user to the deck of the bridge. Vertical lifelines have been introduced that can be used in conjunction with horizontal lifelines and add another dimension to the inspectors range of movement. Factors that must be considered before electing to go with this type of fall protection are: 1. How does the system connect to the bridge? 2. What is the set up time required? 3. Can the inspector set it up or does a manufacturer's rep have to erect the system? 4. If the inspector can set it up, how much training is required? 5. Does the system foul the track? 6. Can the inspector remain connected to the system during train passage? 7. Is the system compatible with a wide range of structures? 8. What does the system cost? 9. Is training included in the cost of the system? These questions will have to be answered by the manufacturer of the system. A system of this type could potentially be very useful on larger scale structures.

Rail rollers/sliders

Rail rollers and sliders are a common means of anchorage employed by bridge and track workers. They provide good mobility when used with welded rail but limit the inspector to track level. On jointed track, joint bars present obstacles that require either removing and replacing the roller/slider or connecting to another roller/slider on the other side of the joint. The sliders are generally much lighter than the rollers and slide surprisingly well on the rail. A major drawback with these devices is that they do foul the track.

Self-Retracting Lanyards

These devices are being used by bridge workers and inspectors on several railroads (see Figure 1). They come in a variety of lengths and can be used in combination with rail rollers, lifelines etc. to give the user a greater range of movement. They use an inertial braking mechanism like a seat belt to arrest the fall. A recent innovation has been to use a self retracting lanyard in combination with roped climbing techniques. This eliminates the need for a belayer (see *two-man roped climbing techniques* for an explanation). An obvious drawback to this technique is that it requires the climber to backtrack in order to retrieve gear placed on the lead and limits the climber to a single pitch. On short spans however, this would not be a problem and the device could be used for improved efficiency.

Binoculars

These are not good for much more than a cursory inspection and should be used on a very limited basis only. One of the major requirements of a good bridge inspection is that the inspector get within arms-length of his subject. With the aging bridge population and the increasingly large number of loading cycles on many of our steel structures, this is becoming more and more critical. Oftentimes a problem area is known to exist on a particular bridge, such as pin movement in a top chord that calls for an increase in inspection frequency. In a case like this, it wouldn't hurt to call the condition to the attention of the local track inspector who could check it periodically with binoculars after the bridge inspector has told him what he should be looking for. This would be an acceptable application.

Rock Climbing Techniques

These techniques offer a great deal of flexibility to the inspector and have recently been adapted for bridge inspection purposes. The advantages of these methods are an ability to reach nearly all areas of a structure with a minimum amount of equipment. Nearly all of the inspectors' gear needed during an inspection can be carried on his person. Costwise, climbing gear is relatively inexpensive, on the order of $1,000 per inspector to get started, and roughly half that each subsequent year to replace worn out or expired equipment. It is not necessary to occupy the track during a climbing inspection although care must be exercised in keeping ropes and webbing from fouling live tracks and other lanes of traffic. A downside to this method of inspection is in the amount of training required. Most railroads will need to contract with a consultant for the initial training but should be able to conduct in-house refresher courses once having gained experience. Also on the downside, the

Figure 1. At Surety Manufacturing's facilities in Edmonton, Alberta, an engineer demonstrates use of a device which "rolls" through cable attachment points without having to disconnect the shock absorbing lanyard.

inspector must be physically fit. Certain techniques will have to be used with some discretion depending on the individual's physical condition, strain and sprain type injuries being the major concern.

Rock Climbing Equipment

Rope

Climbing rope is of a special construction known as kernmantle, which consists of a core (kern) and protective sheath (mantle). It is further grouped into two types, static and dynamic. Static rope is designed to stretch minimally while dynamic rope is designed to stretch about ten percent of its length to absorb the impact forces generated in a fall. A standard climbing rope is 11 millimeters ($\frac{7}{16}$ in.) in diameter and 50 meters (165 ft.) long, although static ropes can often be purchased by the foot.

Harness

Is an arrangement of webbing used to attach a rope, webbing or lanyard to the climber and serves to distribute his weight or the impact forces of a fall for greater comfort and safety. The three major types of harness are seat, chest and full body. The seat harness consists of a waist band and leg loops and distributes the forces of a fall around the pelvic area and is often used with a chest harness. The chest harness is designed to be used with the seat harness and should not be used by itself. Its function is to help keep the hanging climber seated in an upright position. It also gives a would-be rescuer a quick and easy attachment point in the center of the wearer's back. The full body harness functions similarly to the combination seat and chest harness. This type of harness is designed to distribute the forces of a fall over a large area of the body.

Webbing

This is a woven synthetic material that is produced either tubular or flat. The ends of a length of webbing are connected by either tying them (with a water knot) or they are overlapped and stitched. This forms a continuous loop that is useful for anchors, which are called runners or slings. Some runners are constructed of a length of webbing with the ends overlapped and stitched to form eyes. Webbing slings and runners are used with carabiners to allow a quick and easy means of connecting rope and other gear.

Carabiners

These are metal links, usually aluminum or steel, that have spring loaded gates for clipping ropes, webbing and other pieces or equipment. Carabiners come in locking and nonlocking varieties. Rock climbers use the nonlocking type in applications where one hand is often clinging to a hold and the other hand must clip the climbing rope into the carabiner. During this action, the climbers strength is waning and any fumbling or delay can result in a fall. Here, the risk of falling outweighs the risk of carabiner rollout, a situation where the rope unclips itself from the carabiner during a fall by coming across the gate. The bridge inspector, however, has much greater control over his destiny, since a bridge will provide many holds and rest stances (If this is not so, then another form of access must be chosen). For this reason, carabiner rollout proves to be the greater threat to the inspector and locking carabiners are used. Locking carabiners are grouped in two main types, screw barrel and autolocking. The screw barrel variety must be locked manually by twisting the barrel with the fingers until it locks. The autolocking variety has a spring loaded mechanism that automatically locks the carabiner as soon as the gate is released. This is a valuable feature, since it is impossible for the user to forget to lock the carabiner and any movement or pressure against the gate will stand little chance of opening the gate.

Belay Plate

This is a small metal plate with two slotted holes through which a climbing rope passes in order to apply friction. The device is anchored and aids the belayer in arresting the fall of a climber.

Rack

This is a rappel device usually made of aluminum or steel, consisting of a frame with bars that may be added or removed to apply variable friction to the rope during descent. This device is relatively large compared to other rappel devices and is used for long rappels where friction heat buildup can be a problem.

Figure-Eight

This is another type of rappel device, usually made of aluminum or steel used to apply friction to a fixed rope for the purpose of controlling a climbers rate of descent. It has two holes, one for attaching it to the harness, the other through which the rope is passed to apply friction. The figure-eight is usually used for short rappels where friction heat buildup is not a problem.

Prusik

This is a continuous loop of cord used to attach to a fixed rope by means of a prusik knot. The prusik cord is normally of a smaller diameter than the rope it is attached to and grabs when loaded. Uses include: a safety backup for rappelling devices, holding a line in a rescue setup and in pairs for ascending a fixed rope, especially during self rescue.

Ascenders

Are cammed mechanical devices used in pairs that allow a climber to climb a fixed rope. The ascender is easily slid up the rope by the climber but under load tightens on the rope, preventing downward movement.

Rock Climbing Techniques

Two-man roped climbing

This technique, also known as lead climbing, can be used on most structures but is most effective on deck girder type bridges where there is adequate headroom and a bottom lateral system or flanges to walk on (see Figure 2). A dynamic climbing rope connects the two climbers. One man moves at a time, while the other man who is anchored to the structure pays out the rope. This rope handling process is called belaying and the anchored climber doing it is called the belayer. It is the belayer's responsibility to catch his partner if he falls. He is aided by either a mechanical belay device or a special knot that will apply friction to the rope. The inspector who is climbing, known as the leader, places intermediate anchors as he moves along to shorten the distance he could potentially fall. These anchors are made up of webbing slings and carabiners. The climber continues to move and inspect until he reaches a convenient stance where he can set up an anchor and belay. The distance travelled between belay points is called a pitch and is generally less than one rope length. Once the leader has setup an anchor and is ready to belay, roles change. The leader becomes the belayer, and his partner, also known as the second, becomes the climber. As the second climbs toward the belay, he will remove the anchor slings placed by the leader. This process is called cleaning. When the second reaches the belay, he has a choice. He can either continue climbing, at which point he becomes the leader, placing intermediate protection points as he goes, or he can stop, hand over the gear he cleaned from the pitch to his partner and repeat the sequence. Choosing the former results in a leap frog type of progress (the more efficient since gear doesn't have to be passed and a new belay doesn't have to be set up), while the latter resembles an inchworm. This process is repeated as necessary.

Solo Climbing with Webbing

This technique allows an inspector the freedom to move independently with a great deal of mobility and still remain protected. It is limited primarily to truss type bridges. The inspector uses two webbing slings connected to a shock absorber on his harness. One of these slings he wraps once around a structural member and clips it back onto itself with a locking carabiner. This webbing can be slid along the structural member with relative ease and will cinch up tight around the member when weighted for purposes of resting or to arrest a fall (see Figure 3). The second webbing sling is used to move around structural connections to remain continuously protected. This second sling can also be used as a backup anchor if the inspector decides to rest on his primary anchor sling. Although this method is limited primarily to truss type bridges, it can be used on deck girder type structures by prerigging anchor slings from deck level to hang down between the girders. These slings give the inspector something to clip into as he moves along between the girders. A pre-rigged cable is ideal for clipping into and walkway planks or grating also greatly aid the inspector.

Rappelling

This technique allows an inspector to descend a fixed rope. It is most useful on tall structural elements, such as supporting towers that are beyond the reach of an inspection vehicle and where a ground up approach is difficult. The inspector uses a special mechanical device attached to his harness through which the rope passes. This device allows him to slide down the rope and apply friction to control his rate of descent (see Figure 4). One hand acts as the brake controlling the feed of rope through the rappel device. A static rope is preferred for the rappelling operation. A static rope is designed to stretch minimally which gives the rappeller a less bouncy ride and also feeds easier through the rappel device. This static rope is also able to handle the abuse of friction heat buildup associated with rappelling. It is usually best to avoid subjecting your dynamic climbing rope to this

Figure 2. Conrail Bridge Inspector, Tom Gilbert, walks the bottom flange of this deck plate girder bridge. He is tethered with a specialized climbing rope capable of absorbing the impact energy of a fall.

Figure 3. Supervisor Bridge Inspector, Cliff VanWinkle, demonstrates a resting position at this Conrail training session in Columbus, Ohio.

Figure 4. A Burgess & Niple bridge inspector descends fixed ropes to gain access to pin connections on the Perrine Bridge in Idaho.

type of use. There are a few ways of adding redundancy to the rappel. One way is to rappel simultaneously down two ropes independently anchored at the top. Another way is to attach a prusik to the rope above the rappel device, connecting the other end back to an independent locking carabiner on your harness. The prusik will lock up on the fixed rope when weighted, serving to backup the rappel in the unlikely event the rappel device fails. The prusik must be slid along the rope with the free hand to keep it from inadvertently locking up. This common mistake usually happens as a result of not paying enough attention and can be quite frustrating, especially if the knot has gotten out of reach. Another backup method is to place a helper on the ground, who holds the free end of the rope hanging from the rappeller. If he applies force to this free end of rope, this will slow or stop the rappeller just as if the rappeller had applied the force with his brake hand himself. While this technique protects against human error, it does nothing to guard against equipment failure. Probably the most foolproof backup method is the use of an independent belay during rappel. Both of these last two methods impose the burden of requiring an additional man for each rappel setup and may not always be practical depending on available manpower.

Ascending

Refers to climbing a fixed rope. This is done with the aid of either mechanical devices or a specially designed knot, e.g., prusik knot. Because of the exertion required of this method, it is generally not the inspector's first choice. Ascending can sometimes be very useful in regaining your original elevation after rappelling down to a relatively inaccessible location such as the top of a pier. The most popular use of ascending, however, is in self rescue. If a climber has taken a fall, ascenders or prusiks can often be used to regain a stance. Ascending is accomplished by attaching two mechanical ascenders or prusiks to a fixed rope, one above the other. (see Figure 5). The upper ascender is connected to the climber's harness with a webbing sling and locking carabiner while the lower ascender is connected to his foot (or feet). The ascenders are easily slid up the rope but when weighted lock up, preventing any downward movement. By alternately sliding and weighting the ascenders, a climber can move up the rope. This technique takes some practice to use effectively. Most people, when they first try it, wear themselves out before they make much progress and tend to use their arms too much for muscling up the rope. Better results are obtained by relying on the legs for the muscle and using the arms primarily for balance.

Figure 5. A CALTRANS engineer/technician uses mechanical ascending devices to access pin connections on this deck truss highway bridge over the North Fork of the American River in California.

Communication

When two climbers are working together and depending on one another for protection, a simple yet effective means of communicating is essential. Usually the climbers will be within earshot of one another if they are connected by a rope. Oftentimes however, traffic, wind or moving water will interfere with the ability of one inspector to hear the other and radios will become necessary. Small portable units are available although expensive and can be used for this purpose. An added feature that has proven useful is a "bone phone" attachment. This device inserts into and around the ear much like a hearing aid. In addition to receiving, the wearer can transmit by means of a voice actuated device that uses the jawbone to conduct the sound.

A standard set of calls has been in use for many years by climbers, and provides an effective way for the climber to communicate with the belayer. This standard is extremely useful, since it uses short phrases that are easily recognized, even when the call is barely audible. These signals have been carefully devised to communicate common messages and avoid misunderstanding. One set of calls ensures that the belay is set up before the climber moves away from his anchor:

Climber:	"On Belay?"
Belayer:	"Belay On."
Climber:	"Climbing."
Belayer:	"Climb."

You can see the redundancy built into this set of calls. It is very much like asking for a password where two independent responses are required. These calls should be practiced with professionalism in the same way a pilot goes over his preflight checklist. Other common calls used during roped climbing are:

Slack

Used by the climber when he *wants* more rope or slack.

Up rope

Used by the climber when he wants the rope taken in, usually when too much slack has accumulated in the system. A common mistake is to say, "Take Up The Slack." Sometimes background noises will chop off syllables of a communication. In this case, the call "Take Up The Slack" comes across as "- --- -- --- Slack" which conveys the exact opposite meaning intended.

Rescue

This is probably the most difficult area that must be addressed. The FRA requires that prompt rescue be provided. Prompt is left perfectly vague and, at present, there is no precedent to reference. Since every rescue situation will be different, judgment will come into play. Planning and training will prepare the rescuers to act in an organized manner and give them the confidence needed to make decisions and perform under the pressure of an emergency. The first decision that will have to be made is whether or not to seek outside help. If an injury is involved, this decision will be made for you. Radios or cellular phones should be available, their locations and proper use known by every member of the team. Emergency contact telephone numbers for each work location should be obtained in advance, reviewed during the daily job safety briefing and posted conspicuously for ready reference. Unfortunately, not all of these emergency contacts (especially some volunteer groups) are equipped or qualified to perform rescues involving climbing type accidents. And, the situation itself may not allow you the luxury of time needed to bring in rescuers from a distant location. For this reason, some railroads are taking a proactive role in training and equipping their inspectors in carrying out a fall victim rescue themselves.

Rescue training should start with fundamentals, such as how to set up and operate essential equipment. The participants should be instructed in how to evaluate a situation, so that the rescue can be carried out safely, avoiding injury to rescuers and further injury to the victim. Rescue alternatives should be discussed. By far the easiest form of rescue is self rescue. It obviously requires that the victim be conscious and capable of performing the operation. The most simple form of self rescue involves reaching or swinging to a handhold or foothold in order to regain a protected stance. Caution must be exercised. If the system sustained damage during the fall, or if the rope or webbing suspending the fallen climber is in contact with an unprotected sharp edge, then the compromised portion of the system will have to be isolated. This will require assistance. If the system is okay after the fall and the climber can't reach a hold to regain a stance, then ascenders or prusiks can be used to climb up the rope. Prusiks are commonly carried on the person just in case such a need arises. They are light and hardly noticeable compared to mechanical ascenders.

If self rescue is not possible, the two other primary forms of rescue are raising and lowering. Lowering is almost always the easier method to perform but site conditions will often prevent its use. Raising usually requires a mechanical advantage and hence specialized equipment (see Figure 6). Block and Tackle and winches are commonly used for this purpose. These are usually used with a tripod or boom although some can be connected to the structure itself. A boom truck provides a convenient attachment point for rescue equipment. It must be understood, however, that hydraulic or other power operated winches on boom trucks are not manrated and can cause serious injury to a victim during the raise if an obstacle is encountered.

Rescue training should progress over time introducing different rescue scenarios that may be encountered. At more advanced levels, participants should be required to solve problems and make decisions in choosing between alternative methods, rather than just mechanically perform tasks. If "live" victims are used for these exercises, they should always be backed up by an independent belay in the event of a failure.

Figure 6. Bridge Inspector, Craig Judeman, practices a rescue raise during Conrail's Train-The-Trainer Course in Columbus, Ohio.

It would be highly beneficial for anyone who might find themself involved in a rescue to attend a first aid training course. As a bare minimum, it should be pointed out during your rescue training that certain conditions, such as neck and back injuries, can be aggravated by moving the victim. These situations must be handled only by trained medical professionals.

Training

In order for the bridge inspector to take full advantage of the recent revision to FRA Bridge Worker Safety Rules, the railroad—among other things—must provide training in the use of safe climbing procedures (see Figure 7). The details of what to include in this training program are left to the railroad. Also left to the discretion of the railroad is how long and how often training should be conducted in order to keep the inspectors sharp in the use of their techniques. Survey results show that, for climbing techniques, railroads on average put their inspectors through a three day initial course. Since most railroads do not have experience in this area, this initial training is usually left to a consultant. Subsequent interim training in climbing techniques is conducted less than once a year on average, that training lasting between one and one-half and two days per session. Some of the topics being taught in current training programs are:

Three Point Contact

This means that either a hand and two feet or a foot and two hands are in contact with the structure at all times. This was a general rule of thumb for rock and mountain climbers in the early days of the sport when equipment was less forgiving than it is today. Although the sport climbers of today may have abandoned this philosophy in a quest for greater challenge, this basic rule applies well to the work setting where risk must be kept to a minimum and safety remain the number one priority.

Hazards

Look out for and be aware of hazards such as:

- Grease, water, ice, mud or other slippery conditions.
- Sharp or rough edges that could cut or snag you, your clothing or equipment.

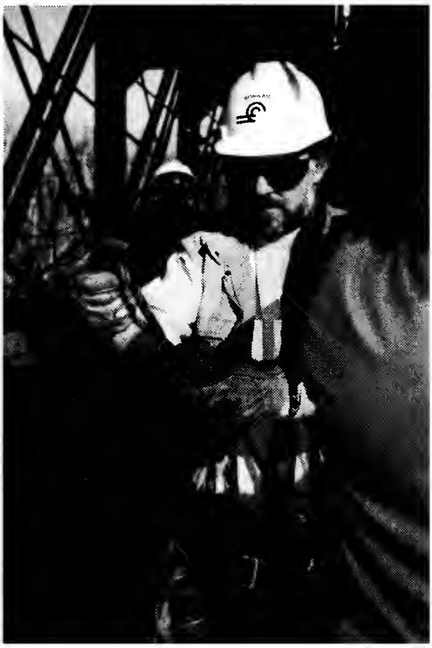

Figure 7. Conrail B&B Supervisor, John Townley, checks knots during a Train-The-Trainer session in Columbus, Ohio.

- Vibrations from moving loads.
- Strong wind.
- Traffic on, over or under structure.
- Animals and insects.

Anchor-Off

Although not an FRA requirement, some railroads are recommending (and some requiring) that, when an inspector has reached a location on a structure where he will remain for an extended period of time or will be engaged in taking measurements, photographs, sketching, etc., he tie off to the structure.

Other notes on training

- *Train with the same equipment that will be used on the job.* If receiving training from a consultant and that consultant is providing the equipment, ensure that the equipment being used meets FRA requirements where necessary and is identical (or nearly identical) to the gear the inspectors will be using on the job.
- *Conduct training on railroad bridges, preferably your own.* Since this is familiar ground and this is where the techniques must apply, it only makes good sense whenever possible. An extension of this idea is to conduct training on one of your large scale structures and combine the training with an actual inspection while the team is assembled. Training can oftentimes be started on the smaller scale approach spans and as the training progresses, so does the size of structure and the exposure to height.

- *Team experienced climbers with new climbers when possible.* This type of on-the-job training will greatly help the new climber develop sound techniques.
- *Use a team approach on large scale structures.* Some inspectors will never be as comfortable with some climbing techniques as others. This is especially true when dealing with extreme heights. Instead of trying to force the issue, try to match individual inspectors' natural abilities with corresponding jobs. Those inspectors better suited to climbing can be placed in that role while others may be better suited in a support function such as ground man, flagman, equipment manager, or vehicle operator, say for shuttling inspectors and equipment. Although there will usually be some healthy tension, or jitters when beginning a large scale inspection at heights, a strong comradery can be born using this team approach.
- *Train two trainers on each territory.* They will become a reliable and familiar source of information when questions arise. They can also rely on each other and use a team approach when conducting training. Also, if one trainer is transferred or bumped out of his position, you stand less of a chance of being left without a qualified trainer.
- *Encourage physical conditioning.* Some railroads have initiated stretching programs in order to cut down on some of the common strain/sprain type injuries sustained on the job. Climbing places a greater physical demand on the inspector than many activities. Some of the techniques will have to be used with discretion depending on the individual's physical condition. An overall fitness program addressing strength, endurance and flexibility will greatly reduce the likelihood of sustaining an injury while performing climbing techniques.

Conclusion

The equipment and techniques available today will undoubtedly make possible a safer and more efficient inspection than was possible in the past. Proper training will ensure the inspector can conduct his job safely, by arming him with the knowledge and skills necessary to choose between alternative systems and then use that chosen system to its best advantage. Proper planning in the event of an accident and practice in the use of rescue procedures will give the inspector the confidence needed to do a quality job and then return safely to his family when that job is done. Even though we have made great strides in protecting our bridge inspectors, a false sense of security must not be developed. Every inspector must regularly check his gear for damage, wear and proper set up. He must also remain alert and aware of the consequences of a fall. There are no shortcuts or pat answers. You can't simply tell an inspector, "Don't look down." His eyes are his tools and he still has to inspect that bottom chord. What you can do is provide him with the skills, knowledge and equipment necessary to perform a safe, thorough and efficient inspection.

Appendix A
Survey Response—1995 ARBBA Special Subject No. 2 "Fall Protection and Safety Climbing Devices for Bridge Inspection"

Responses:

Major (Class I's & Passenger)	Surveyed: 16	Responses: 11	Response Rate: 69%
Reg. & Short Lines:	Surveyed: 22	Responses: 5	Response Rate: 23%
Total	Surveyed: 38	Responses: 16	Response Rate: 42%

1. Number of Bridges

Responses shown in *italics*.

Undergrade Bridges .... ________

12,361; 310; —; 1,165; 5,500; —; 5,000; 9,421; —; 2,421;
—; 336; —; 40; 265; 587

Overhead Bridges .... ________

3,158; 75; —; —; 618; 10,109; 200; 1,992; —; 456;
—; —; —; 78; 5; 116

Total Bridges:

15,519; 385; 10,400; 1,165; 6,118; 10,109; 5,200; 11,413; —; 2,877;
—; 336; —; 118; 270; 703;

2. Frequency of First Level* Bridge Inspections

*First Level is defined as a regularly scheduled, hands-on inspection; does not include special detailed steel, timber, or underwater inspections.

What is the frequency of first level bridge inspections? (Please provide explanation; continue on back.)

Annually(8)   Annually with special interim inspections (5)   Semiannually (3)*

** Throughout this report, numbers in parentheses will indicate the number of railroads responding affirmatively*

I. Who Performs First Level Bridge Inspection?

A. Contracted Service.. ________%

If 100%, go directly to Part IV, Access Methods.

0; 0; 0; 0; 0; 1; 0; 2; 0; 0;
0; 0; 50; 0; 0; 0

B. In-House.. ________%

Total percentage Part A plus Part B should equal 100%.

100; 100; 100; 100; 100; 99; 100; 98; 100; 100;
100; 100; 50; 100; 100; 100

1. Number Of First Level Inspectors: .... ________%

47; 3; 9; —; 12; 21; 2; 33; 28; 2;
1; 2; 1; 2; 1; 8

Bridges per Inspector:

330; 128; 1,156; —; 510; 481; 2,600; 346; —; 1,439;
—; 168; —; 59; 270; 88

Mean: *631 br's/insp*
Mean (drop hi, lo): *492 br's/insp*
Median: *338 br's/insp*

2. Description of First Level Inspectors:
Check applicable box(es).
a) Union Affiliated?
☐ Agreement Employee *(9)*
☐ Non-Agreement Employee *(9) three respondents have both, one no response*

b) Title
☐ Supervisor *(9)*
☐ Inspector *(11) four respondents have both*
☐ Other, please explain: *one project engineer, one engineer, one assistant supervisor*

3. Inspection Type
Should total 100%

a) Individual.. ________%

80;	*15?;*	*67;*	*90;*	*75;*	*80;*	*0;*	*6;*	*0;*	*0;*
	25;	*50;*	*0;*	*100;*	*0;*	*0*			

b) Two-man.. ________%

19;	*60?;*	*0;*	*10;*	*25;*	*18;*	*99;*	*49;*	*75;*	*100;*
	75;	*50;*	*100;*	*0;*	*0;*	*100*			

c) Team.. ________%

1;	*15?;*	*33;*	*0;*	*0;*	*2;*	*1;*	*45;*	*25;*	*0;*
	0;	*0;*	*0;*	*0;*	*100;*	*0*			

II. Access Methods

Check applicable box(es).
☐ Rappel *(3)* ☐ Belay *(3)* ☐ Solo Climb *(10)* ☐ Steel Skates *(2)*
☐ Insp. Vehicle *(7)* ☐ Scaffolding *(7)* ☐ Ladder *(14)* ☐ Net, Climbing *(1)*
☐ Lifelines *(12)* ☐ Binocular *(12)* ☐ Boat *(8)*
☐ Other, please explain: *walk thru, walk under, boom truck & basket*

III. Equipment

Check applicable box(es).

A. Mechanized

I. ☐ Inspection Vehicle

a) ☐ Hi-rail *(9)*

if yes, how many vehicles?

1;	*1;*	*1;*	*1;*	*1;*	*1;*	*1;*	*5;*	*0;*	*0;*
	0;	*3;*	*0;*	*0;*	*0;*	*0*			

Please describe: *ready for replacement, borrow as needed*

b) ☐ Hiway *(2)*

if yes, how many vehicles? ______

0;	*0;*	*0;*	*0;*	*0;*	*0;*	*0;*	*8;*	*0;*	*0;*
	0;	*0;*	*0;*	*0;*	*0;*	*4*			

Please describe: 8 *pickups*

2. ☐ Cherry Picker

a) ☐ Hi-Rail *(3)*

if yes, how many vehicles? ______

0; 0; 0; 0; 0; 75; 1; 0; 0; 0;
0; 0; 0; 0; 0; 0; 0; 1; 0

(75 hi-rail boom trucks w/capability of man bucket on boom tip.)

b) ☐ Hiway *(1)*

if yes, how many vehicles? ______

0; 0; 0; 2; 0; 0; 0; 0; 0; 0;
0; 0; 0; 0; 0; 0

3. ☐ Scissor Platform

a) ☐ Vehicle Mounted *(1)*

if yes, how many vehicles? ______

0; 0; 0; 0; 0; 0; 1; 0; 0; 0;
0; 0; 0; 0; 0; 0

b) ☐ Tag-Along *(1)*

if yes, how many vehicles? ______

0; 0; 0; 0; 0; 0; 0; 0; 0; 0;
0; 0; 1; 0; 0; 0

4. ☐ Spider Buckets *(1)*

if yes, how many? ______

0; 0; 0; 0; 0; 0; 0; 9; 0; 0;
0; 0; 0; 0; 0; 0;

☐ Other, please explain: *Rent above as needed*

B. Temporary Nonmechanized Platform Access

1. ☐ Ladders *(12)*
2. ☐ Scaffolding *(6)*
3. ☐ Staging *(3)*
4. ☐ Rope Ladder *(1)*
5. ☐ Etrier, webbing loop ladder *(2)*
6. ☐ Personnel Net *(1)*
7. ☐ Other, please explain: *climbing gear, see below*

C. Personal Equipment

1. Harness
   a) ☐ Full Body *(12)*
   b) ☐ Seat Harness *(2)*
   c) ☐ Seat Harness w/Chest Harness *(1)*
   d) ☐ Other, please explain:

2. Ropes
   a) ☐ Kernmantle (rock climbing type, core & sheath)
      (1) ☐ Static *(4)*
      (2) ☐ Dynamic *(3)*
   b) ☐ Hawser Laid, conventional rope *(3)*
   c) ☐ Other, please explain:

3. Retractable Lanyards/Lifelines *(7)*
   Please describe length, etc.: *between 4' and 85'*

4. Rappel Hardware
   a) ☐ Rappel Rack *(2)*
   b) ☐ Figure Eight *(2)*
   c) ☐ Carabiner Brake *(1)*
   d) ☐ Carabiner w/Munter Hitch *(1)*
   e) ☐ Other, please explain: *brand name specialty device*
5. Belay Hardware
   a) ☐ Belay Plate *(2)*
   b) ☐ Figure Eight *(2)*
   c) ☐ Carabiner w/Munter Hitch *(1)*
   d) ☐ Other, please explain: *brand name specialty device*
6. Carabiners
   a) ☐ Non-Locking *(0)*
   b) ☐ Locking
      (1) ☐ Screw barrel *(4)*
      (2) ☐ Spring Loaded Auto-locking *(8)*
      (3) ☐ Other, please explain:
7. Webbing
   a) ☐ Continuous Loop Sewn Slings *(7)*
   b) ☐ On roll, tie your own slings *(1)*
   c) ☐ Other, please explain:
8. Shock Absorbing Devices
   a) ☐ Rip Stitch *(15)*
   b) ☐ Other, please explain: *integral to life line*
9. Head Protection
   a) ☐ Standard Construction Type Hard Hat *(15)*
   b) ☐ Rock Climbing Helmet *(1)*
   c) ☐ Other, please explain:
10. Shoes
    a) ☐ Steel Toe Boot *(16)*
    b) ☐ Other, please explain: *steel sole plate*
11. Gloves
    a) ☐ Ordinary leather (or similar) work glove *(16)*
    b) ☐ Other, please explain: *fingerless, cotton w/leather palm, goatskin fingers w/cowhide palm*

D. Rescue Equipment
Check applicable box(es).
1. ☐ First Aid Kit *(15)*
2. ☐ Hand held Radios *(13)*
3. ☐ Boom Truck *(9)*
4. ☐ Block & Tackle System *(7)*
   Capacity (one-man/two-man/other)? *one man, two man*
5. ☐ Hoisting Winch *(6)*
   Capacity (one-man/two-man/other)? *two man*
6. ☐ Rescue Tripod *(5)*
   Capacity (one-man/two-man/other)?
7. ☐ Track mounted boom/derrick *(5)*
   Capacity(one-man/two-man/other)?
8. ☐ Ladder Derrick *(0)*
   Capacity (one-man/two-man/other)?

9. ☐ Other, please explain: *self rescue, ladder rescue*
How is the rescue equipment secured to the bridge? *w/webbing to tie or tie; vehicle w/rail dogs; rail; tie; top flange girder; tie downs*

E. Over Water Gear
Check applicable box(es).
a) ☐ Life Vest *(12)*
b) ☐ Ring Buoys *(9)*
c) ☐ Skiff *(10)*
d) ☐ Other, please explain: *use fall arrest instead*

IV. Permanently Affixed Appurtenances
Check applicable box(es).

A. ☐ Inspection Walkway
1. ☐ In place *(12)*
2. ☐ Plan to install *(5)*

B. ☐ Inspection Cable
1. ☐ In place *(10)*
2. ☐ Plan to install *(8)*

C. ☐ Inspection Angle or Pipe (Attached near top of abutment for bridge seat and bearing inspection; used as a foothold, handhold, and/or anchor)
1. ☐ In place *(2)*
2. ☐ Plan to install *(0)*

D. Ladders
1. ☐ In place *(11)*
2. ☐ Plan to install *(3)*

E. Grab Irons
1. ☐ In place *(7)*
2. ☐ Plan to install *(1)*

F. Inspection Trolley
1. ☐ In place *(3)*
2. ☐ Plan to install *(0)*

G. Eye Bolts
1. ☐ In place *(2)*
2. ☐ Plan to install *(2)*
☐ Other, please explain: *embedded tie-offs in new pier construction (1), plans to upgrade accessibility of movable bridges (1)*

V. Fall Protection Training

A. Do bridge inspection personnel receive specialized fall protection training, different than that of the bridge worker? .... ☐ Yes *(10)* ☐ No *(6)*

B. Initial Training
Duration of Initial Training (days)

3; 0.5; 1; 1; 1; 1; 0.5; 4; 5; 5;
1; 1; 0.5 0.5; 0.5 3

Overall (16):	*Mean: 1.78*	*Mean (w/o hi, lo): 1.64*	*Median: 1*
Climbing techniques (3):	*Mean: 3.33*	*Mean (w/o hi, lo): 3*	*Median: 3*

C. Interim/Refresher Training
1. Frequency of Interim Training (times per year):

1; 2; 0 ; 1; 1; 1; 1; 0.5; 2; 0.5
0; 0; 1; 1; 0; 0.5

Overall (16):	*Mean: 0.78*	*Mean (w/o hi, lo): 0.79*	*Median: 1*
Climbing techniques (3):	*Mean: 0.67*	*Mean (w/o hi, lo): 0.5*	*Median: 0.5*

2. Duration Of Interim Training (days)

3; 0.5; —; 1; 0.5; 0.5; 0.5; 0.5; 1; 3;
—; —; 0.5; 0.5; —; —

Overall (11):	*Mean: 1.04*	*Mean (w/o hi, lo): 0.89*	*Median: 0.5*
Climbing techniques (2):	*Mean: 1.75*	*Mean (w/o hi, lo): na*	*Median: na*

D. Topics:
FRA Rules, company policy, rescue, equipment care, climbing techniques, physical fitness, general safety, ladder use, anchorage, first aid

E. Type of Training
Should total 100%.

1. Classroom.. ________%

15; 0; 50; 50; 50; 50; 60; 50; 30; 60;
0; 75; 100; 80; 50; 33

Overall (16):	*Mean: 47*	*Mean (w/o hi, lo): 47*	*Median: 50*
Climbing techniques (3):	*Mean: 33*	*Mean (w/o hi, lo): 33*	*Median: 33*

2. Field.. ________%

85; 100; 50; 50; 50; 50; 10; 25; 65; 40;
100; 25; 0; 20; 50; 67

Overall (16):	*Mean: 49*	*Mean (w/o hi, lo): 49*	*Median: 50*
Climbing techniques (3):	*Mean: 59*	*Mean (w/o hi, lo): 67*	*Median: 67*

3. Simulated Field.. ________%

0; 0; 0; 0; 0; 0; 30; 25; 0; 0;
0; 0; 0; 0; 0; 0

Overall (16):	*Mean: 3*	*Mean (w/o hi, lo): 2*	*Median: 0*
Climbing techniques (3):	*Mean: 8*	*Mean (w/o hi, lo): 0*	*Median: 0*

4. Other, please explain: *demonstrations* ________%

0; 0; 0; 0; 0; 0; 0; 0; 5; 0;
0; 0; 0; 0; 0; 0;

Overall (16):	*Mean: 1*	*Mean (w/o hi, lo): 0*	*Median: 0*
Climbing techniques (3):	*Mean: 0*	*Mean (w/o hi, lo): 0*	*Median: 0*

F. Training Materials
Check applicable box(es).

1. ☐ Slides *(9)*
2. ☐ Videos *(11)*
3. ☐ Written Text *(15)*
4. ☐ Multi-Media *(1)*
5. Testing Instrument(s)
   a) ☐ Written Test *(7)*
   b) ☐ Demonstration *(10)*
   c) ☐ None *(3)*
   d) ☐ Other, please explain: *(0)*

G. Contracted Training Service (☐ Sought, but unable to locate training; Go directly to In-House Training, Section H)

1. Credentials of Consultant
   Check applicable box(es).

a) ☐ Training Manager is Licensed P.E. *(3)*
b) ☐ Other, please explain: *Training by manufacturer/supplier (4)*

2. Experience of Consultant (years)

5+; —; —; 4; 5+; —; —; 5+; —; 5+;
—; 5+; —; —; —; 5+

3. Approximate Average Annual Cost Per Person For Contracted Training.
Initial Training: $_______

1,000; —; —; —; 695; —; —; 500; —; —;
—; 3,000; —; —; —; 1,350

Overall (5):	*Mean: 1,309*	*Mean (w/o hi, lo): 1,015*	*Median: 1,000*
Climbing techniques (3):	*Mean: 950*	*Mean (w/o hi, lo): 1,000*	*Median: 1,000*

Interim/Refresher Training: $_______ *no responses*

4. Level of Satisfaction: E=Excellent, G=Good, F=Fair, P=Poor

G; —; —; G; E; —; —; F; G; E;
—; G; —; —; —; G

H. In-House Training
1. Who conducts training?
☐ Safety Department *(4)*
☐ M/W Department *(16)*

2. Would you be willing to share your training program with other railroads?
☐ Yes *(6)* ☐ No *(6)* *No response (6)*

VI. Rescue

A. Do you have written rescue procedures? .... ☐ Yes *(8)* ☐ No *(8)*

B. If yes, please briefly list the major points addressed:
Raise/lower victim, emergency medical, emergency contacts, hang time, step by step rescue, who is in charge

C. Do you conduct practice rescue drills? ☐ Yes *(8)* ☐ No *(6)* *No response (2)*
If yes, please describe: *Use dummy, use man on separate belay.*

VII. Problems

Please describe any fall protection problems encountered associated with bridge inspection: *Stringer-Floorbeam connection access, fall protection while climbing endpost, suitable head protection, creosote getting on synthetics, individuals unwilling to employ techniques, rescue, one-man rescue, lack of permanent cables, corroded grab rods, damaged access ladders, horizontal movement, difficulty accessing some areas of bridge, legal aspects, need approved curriculum for trainers.*

VIII. Industry Developments

A. How do you keep updated on industry standards and developments?
☐ Sales personnel *(12)*
☐ Magazines *(14)*
☐ Word of Mouth *(11)*
☐ Other, please explain: *AREA, ARBBA, Safety Officer, Federal Register*

B. Is there a need for:
☐ A permanent Fall Protection Committee: *Yes (6)* *No (1)* *No response (9)*
☐ A Fall Protection Resource or Newsletter: *Yes (11)* *No (1)* *No response (4)*
using B&B members, Suppliers, etc?

APPENDIX B—GLOSSARY OF ROCK CLIMBING TERMS

aid climbing: Leaning, hanging, or otherwise pulling on the rope or *anchor* in order to rest or make progress, as distinguished from *free climbing.*

anchor: The means of attaching the climber to the structure. *Webbing* and *carabiners* are most often used for this purpose.

ascend: The process of climbing up a *fixed rope* using *ascenders* or *prusiks.*

ascender: A cammed mechanical device or special knot that allows the climber to climb a *fixed rope.* The ascender is easily slid up the rope by the climber but under load tightens on the rope, preventing downward movement. Normally ascenders are used in pairs, with the lower used in conjunction with a foot loop, and the upper attached to the climbing harness.

belay: Paying out or taking in rope in a manner that will allow one climber, the *belayer,* to arrest a fall by another climber. Either a mechanical device, a specifically designed knot or the belayer's body are used to apply friction to the climbing rope to help arrest the fall.

belay plate: A small metal plate with two slotted holes through which a climbing rope passes in order to apply friction.

bight: A knot tying term that means a loop or bend made in a rope. Also see *on-a-bight.*

bowline: A knot commonly used to attach the climbing rope to the *harness.* The *figure-eight-on-a-bight,* however, has become the more popular knot for this use.

braiding: A method of tying webbing in a series of slip knots so that it's length is shortened for carrying.

butterfly knot: This is a knot usually tied in the middle of a rope. It is often used to attach a third climber to a mid-point in the rope and behaves well when loaded in three directions.

carabiner: A metal link, usually aluminum or steel with a spring-loaded, autoclosing gate that permits insertion of the climbing rope webbing sling or lanyard used mainly for attaching the climber to *anchors.* They come in the locking and non-locking variety.

carabiner brake: A device producing friction on the rope assembled from three or more carabiners, used mainly for rappelling. This device places stresses on the carabiners for which they were not designed and should not be used.

chest harness: A harness made of synthetic webbing that is used with a seat harness to attach the climber to a rope, webbing, or lanyard. The chest harness helps in keeping a fallen climber in an upright hanging position.

clean: To remove all the anchors placed on a *pitch* by the leader. This is usually done by the *second* or subsequent person as he climbs.

clove hitch: A simple knot that can be tied quickly that is sometimes used for *anchoring* and *belaying.* It is not recommended as the primary knot in an anchor. The *figure-eight-on-a-bight* is preferred for this purpose. The *clove hitch* is useful as a secondary knot in an anchor as it can be adjusted quickly and easily in order to equalize the load on the *anchor.*

coiling: A method of storing a climbing rope. Although *coiling* is neat, if the rope is not properly *stacked* into a loose pile before climbing is begun, knots and kinks tend to form when the rope is payed out. A recent development is the use of rope bags for storing ropes uncoiled in a loose fashion eliminating the need to *coil* or *stack.* Caution should be exercised to ensure that proper inspection of the climbing rope be conducted and that ropes are not stored wet.

double fisherman's knot: A very secure knot used to join two ends of rope, also known as a double *grapevine knot.* It can also be tied in a triple, etc. manner for additional security.

double grapevine knot: A very secure knot used to join two ends of rope, also known as a *double fisherman's knot*. It can also be tied in a triple, etc. manner for additional security.

dynamic rope: A rope designed to stretch in order to absorb the impact energy of a fall, distinguished from *static rope*. Made of *kernmantle* construction, a *dynamic rope* normally stretches around ten percent of its paid-out length.

etrier: Pronounced ay-tree-ay, is a piece of webbing with loops placed at intervals for steps and is used for *aid-climbing*.

figure-eight: A *rappel* device used to apply friction to a *fixed rope* during descent. Made of metal, usually aluminum or steel, it has two holes, one for attaching it to the *harness* and the other through which the rope is passed to apply friction. The *figure-eight* is usually used for short *rappels* where heat buildup is not a problem.

figure-eight-on-a bight: The most popular knot for attaching the climbing rope to the *harness*. This knot is also used to attach the climbing rope to *carabiners* at *anchor* locations and can be tied directly with the end of the rope doubled or tied in a follow-through fashion doubling the end of the rope back through the single line knot. This knot is often backed up with a double (or triple, etc.) *overhand knot*. This serves to provide an additional margin of safety as well as neatly tuck away that extra length of rope hanging out the end of the knot.

fixed rope: A rope that is securely *anchored* from above and most often used for *ascending* or *rappelling*.

free climbing: *Roped climbing* using only the *holds* the structure itself provides, using the rope and *anchors* for fall protection only. The climber does not lean, stand or pull on them, as distinguished from aid *climbing*.

free solo: *Free climbing* without the protection of a rope and *anchors*.

full-body harness: A *harness* made of synthetic *webbing* used to attach a rope, *webbing* or lanyard to the climber. This harness is designed to spread the forces generated in a fall over a large portion of the body and to keep the fallen climber in an upright hanging position.

harness: An arrangement of synthetic *webbing* used to attach a rope, *webbing* or lanyard to the climber and serves to distribute his weight or the impact forces of a fall for greater comfort and safety. See also *full-body harness*, *seat harness* and *chest harnesses*.

hawser laid: A conventional method of rope construction in which fibers are twisted into three or four strands which are then twisted into the rope. This type of rope was superseded by *kernmantle* ropes in 1953 for climbing purposes.

hold: An irregularity in a surface that can be used by a climber to either grab with a hand or place a foot upon in order to rest or progress.

kernmantle: A type of rope construction generally used for climbing-ropes consisting of a protective sheath (mantle) and core (kern).

lead climbing: Also called roped climbing, this is a traditional form of fall protection used in rock climbing and mountaineering. Climbers connected by a rope alternately protect one another as one climbs placing intermediate anchors and the other *belays*.

leader: The climber leading a *pitch* who first places the *protection slings* that must be removed later by the *second* or subsequent climbers.

locking carabiner: A *carabiner* with a locking gate. These come in two varieties, one with a screw barrel that must be manually twisted to lock the gate; the other with a spring-loaded autolocking gate that closes and locks when released.

munter hitch: A special knot used with a locking carabiner to apply friction to a rope for *belaying* or *rappelling*. It is very useful since it can be used in the event a *belay* or *rappel* device is dropped or lost during a climb.

on-a-bight: A suffix often added to the name of a knot to denote that the knot was tied with the end of the rope doubled in order to form a loop or *bight* in the end of the rope.

overhand knot: This knot has limited use in climbing applications. It is used primarily to back up certain knots such as the *figure-eight-on-a-bight* or the *bowline* and is often tied doubled or tripled for this purpose. Tied *on-a-bight* so that a loop is formed, it is sometimes used by the uninitiated to tie into a *harness* or for *anchoring*, however, the *figure-eight-on-a-bight* is a much better choice for this use since it is stronger and less likely to jam under load.

pitch: One section of a *roped climb* between two *belay* points, not usually longer than one rope length.

protection: Slings placed by the *leader* for the purpose of shortening the length of a fall. This term is often just called *pro*.

prusik: A continuous loop of cord used to attach to a fixed rope by means of a *prusik knot*. The *prusik* cord is normally of a smaller diameter than the rope it is attached to and grabs when loaded. Uses include: a safety backup for *rappelling* devices, holding a line taught in a rescue setup, and used in pairs for *ascending a fixed rope*, especially during self rescue. This term is also used as a verb to describe the action of *ascending a fixed rope*.

prusik knot: A special friction knot for attaching a *prusik* to a rope.

rack: A rappel device usually made of aluminum or steel consisting of a frame with brake bars that may be added or removed to apply variable friction to the rope during descent. This device is relatively large compared to other rappel devices and is used for long rappels where friction heat buildup can be a problem.

rappel: To descend a *fixed rope* aided by the application of friction by a specially designed *rappel* device, knot, or by systematically wrapping the rope around the climber's body. Padding is usually needed when the latter form is used.

rope drag: Frictional resistance of the rope passing through *carabiners* in a system. This drag impedes the movement of a climber and must be kept to a minimum by choosing *anchor* locations and *runner* lengths that will allow the rope to run in as nearly a straight line as possible.

roped climbing: Also called lead climbing, this is a traditional form of fall protection used in rock climbing and mountaineering. Climbers connected by a rope alternately protect one another as one climbs placing intermediate anchors and the other *belays*.

runner: Usually a continuous loop of webbing used as an *anchor* in conjunction with a *carabiner*. It can also be in the form of a length of *webbing* with ends doubled back and stitched to form an eye on each end.

seat harness: A *harness* made of synthetic *webbing* consisting of a waist band and leg loops used to attach the climber to a rope, *webbing* or lanyard and designed to distribute the forces of a fall around the pelvic area. The *seat harness* is often used with a *chest harness*.

second: A term applied to the climber following the *leader* who must usually remove the protection *slings* placed by the *leader*.

sling: Usually a continuous loop of webbing used as an *anchor* in conjunction with a *carabiner*.

solo climbing: Climbing alone either with a self *belay* or *free*.

stack: To place the rope in a pile so that it will pay out without tangling or kinking.

static rope: A rope, either *hawser laid* or *kernmantle*, designed to stretch minimally, distinguished from *dynamic rope*. Usually used in *fixed rope* applications such as rappelling or ascending where a large amount of stretch is undesirable, a *static rope* usually stretches at or less than two percent of its payed out length.

traverse: To move or climb without gaining or losing altitude, generally describing horizontal movement. The term can also be used as a noun to describe a horizontal section of a climb.

water knot: The only suitable knot for tying flat objects such as webbing or leather. This knot looks much like an *overhand on-a-bight* but must be tied in a follow-through fashion so that the ends extend out opposite sides of the knot.

webbing: A woven synthetic material that is produced either tubular or flat. *Webbing* has a variety of uses but is most commonly used for anchors in the form of *runners* or *slings*.

RISK SHARING IN TUNNEL PROJECTS

By: Thomas R. Kuesel*

Tunneling has been described as a process started by geologists and completed by lawyers. The risks uncovered by the geologists are of interest to engineers and contractors. Those uncovered by the lawyers also engage the attention of owners, and are the subject of this paper.

The classic case involved a water supply tunnel for the city of Aurora, Colorado. The low bid came from a contractor named Lindeman. The City had never heard of him, but he had a reputable surety and an acceptable record, so they gave him the contract. From the moment of Notice to Proceed, the contract turned into a dogfight—arguments all day long, claims every other day, and general uproar. Halfway through the mountain the contractor announced "I'm pulling everyone off the job. We'll see you in court." Whew! "At least we're rid of that guy—let the lawyers deal with him."

But the City still needed the water, so they put together a new contract, for completion of the tunnel. They got three bids. The low bid came from their friend Lindeman, who offered to sue them again if they didn't give him the contract.

How did tunnel construction get to be such a contentious business, and what can be done about it? Some historical perspective may be helpful. Construction contract law is generally based on the underlying premise that a contract is an agreement between two parties in which one party undertakes to perform for the other a definite, mutually understood task for a defined compensation. The American system of competitive bid contracting is based on the assumption that the work to be performed, and the conditions under which it is to be performed, are perfectly and unambiguously defined in the contract documents, so that a simple comparison of bid prices is sufficient to determine a fair selection, and contract award is made to the lowest responsive bidder.

This ideal is rarely reached in any construction contract, but it is especially illusory for tunnel (or any underground) construction, owing to several peculiarities inherent in underground work:

- Tunnels are invariably lengthy structures, and over extended lengths the characteristics of the ground may vary widely and unpredictably. Despite the most comprehensive geotechnical investigations, the exact nature of the ground is never completely disclosed until it is exposed by excavation at the tunnel heading. As a result, the work that is actually performed may differ in small or large measure from that expected by either or both parties at the time of contract award.
- The methods, equipment, and skills required for safe and economical tunnel construction depend on the nature of the ground, and may be disproportionately sensitive to small changes in ground characteristics.
- Preconstruction investigations can determine the characteristics of (more or less representative samples of) the existing ground. The processes of construction may change these characteristics (e.g., destressing rock joints by excavation may cause them to open, and convert a dry tunnel into a waterfall). The contractor's choice of equipment and construction methods, and the skill of his workers, may increase or decrease deleterious changes in ground characteristics during construction.
- In urban tunnel work, the existence and location of unknown buried obstructions and hazardous conditions (such as gasoline derived from abandoned leaking underground storage tanks) is difficult to determine beforehand, and can have major effects on the work.

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- In urban work, it is not uncommon that the cost of dealing with third parties (government regulations, adjacent structure protection, support and relocation of utilities, hauling and disposal of excavated materials, traffic maintenance, site clearance and preparation, and restoration of the surface after completion of construction) may equal or exceed the cost of constructing the desired facility. The best of agreements between two parties regarding how third parties will act or behave is considerably less than infallible.

In traditional construction contracting practice, the owner allocates all risks to the contractor. The contractor's bid price was supposed to cover the costs of mitigating all problems related to unknown site or geotechnical conditions, as well as all delays or difficulties introduced by the actions or omissions of third parties. These were essentially gambles, which the owner asked the contractor to make.

But at the same time, the owner told all bidders that the contract would be awarded to the lowest responsive bid. A bid that included reservations or exclusions regarding any risk was deemed "non-responsive."

This practice produced two results:

1. Experienced and prudent contractors, who included in their bids substantial contingencies to cover perceived risks, found that their bids were rarely low. In the minority of contracts that they did win (generally the more difficult ones that scared inexperienced contractors away), they found that if the risks did not materialize they made a windfall, and if they did materialize they went broke. In this boom-and-bust climate, if the boom came first the contractor survived, and if the bust came first, he went out of business. Tunnel construction contracting was a short-lived occupation.

2. Less experienced, or more adventurous, contractors took an optimistic view of risks of underground construction, and included little or no contingencies in their bids. They won more contracts, and when the risks materialized, they mounted a vigorous campaign of claims and litigation, generally on the assertion that the contract was defective, in that the owner knew, or should have known, or failed to take adequate measures to discover, conditions of which the contractor should have been advised in the solicitation of bids.

This was the situation on the 1950's, when a great surge in urban underground construction occurred across the United States. Construction litigation became an increasingly popular and lucrative occupation (for the lawyers), and tunnel construction developed a bad name among owners and the general public, as prone to large, unexpected overruns of projected costs and schedules.

The first attempt to mitigate these problems through improvement in the standard form of contract came from the Corps of Engineers, which developed a "Differing Site Conditions" clause to deal with arguments about whether or not encountered conditions were unexpected. This establishes contractual recognition that actual conditions *might* differ from those expected, and requires the owner to investigate and make a determination, and to make an "equitable adjustment" to the contract if a difference is upheld.

The Differing Site Conditions clause enabled the owner's Contracting Officer to make equitable contract adjustments for the more egregious cases, where he could in good conscience find that site conditions did indeed differ so clearly that he could not, by accepting the contractor's claim, be held to breach his duty to the owner, to protect his interests. General acceptance of this clause by the courts eased the strain of loosening the rigid contract bonds, and the climate for tunnel construction improved. Prudent contractors found that they were protected from the more outrageous risks, their contingencies came down, and they started to gain more contracts. Owners found that the quality of their tunnel contractors improved, and their litigation became less raucous.

Nonetheless, many cases remained in which the owner and the contractor did not agree on whether the conditions differed, and these cases still ended up in the court.

It became apparent that tunnel construction is an arcane art to the legal profession, and much of the time and cost of litigation was being expended on educating the lawyers and judges on the technical terms and practicalities of tunnel construction. Since this education was perforce rudimentary, the lawyers and judges tended to try to find grounds for deciding the case on the basis of fine points of law and alleged legal precedents (which they understood), rather than what was practicable, realistic, and effective in the tunnel heading (which they did not understand).

This situation satisfied neither the owners nor the contractors. A particular burden was the length of time the litigation process consumed, which meant that final settlement was frequently delayed until years (sometimes many years) after construction was completed.

The next major improvement in contract form came with the development of the Disputes Review Board procedure, which grew out of the Eisenhower Tunnel project in Colorado, for which A. A. Mathews recommended a Mediation Board in the 1970's.

A Disputes Review Board (DRB) is composed of three members, all of whom are experts in tunnel construction or tunnel engineering. One member is selected by the owner, one by the contractor, and the third by the first two. The Board Members provide informal mediation of technical and contractual issues on which the owner and contractor are unable to reach agreement under the provisions of the contract. The Board provides a written report and recommendation regarding each dispute, which is not binding, but carries great weight because the members are chosen, and recognized, for their professional experience and perspective. If either party rejects the Board's recommendation, it is with the knowledge that in subsequent litigation, the courts will value the Board's recommendation highly.

The Disputes Review Board procedure has been found to have a number of advantages for both the owner and the contractor.

- The DRB procedure is much less costly and time-consuming than formal litigation.
- Recommendations of professional experts are more likely to be based on practical considerations than on abstruse points of law.
- The saving in senior management time devoted to contract dispute resolution is significant.
- Disputes are settled promptly while the construction continues to go forward, and consequential delays and costs are reduced.
- The process is much less adversarial than litigation, and the climate of contract administration is improved.

The DRB process has also been found to have some unexpected benefits:

- By its very existence, the Board reduces the incidence of claims and fosters settlement between the two parties, because both parties know that the Board cannot be bluffed and that insecure claims (or arbitrary disallowances) are likely to be rejected by the Board.
- Since the Contracting Officer is relieved of the onus of being both the owner's representative and the judge of the contractor's claims, he is able to be more flexible in dealing with unanticipated developments during construction. Since he can pass ambiguous issues to the Board for equitable resolution, he does not have to find black and white solutions to gray problems.

With respect to the largest classification of tunnel construction contract disputes, Differing Site Conditions, an important companion to the DRB procedure is the Geotechnical Design Summary Report (GDSR). The GDSR, which is a part of the contract documents, is a record of the Engineer's preconstruction site investigations and laboratory tests, as well as a discussion of the Engineer's interpretation of how this information has affected the design and may affect construction. In particular, the GDSR is intended to illuminate any restrictions on construction methods, equipment, or sequences that may be included in the specifications.

In broad terms, the GDSR is intended to define a basis for the solicitation of bids. It provides the assumptions with respect to site geotechnical conditions that are to be used by both the contractor and the owner in performance of their respective duties under the contract. The GDSR defines a box—if the actual conditions disclosed during construction fall within this box, they are covered by the contract; if they fall outside the box, they require modification to the contract.

In cases of disagreement about whether a conditions falls inside or outside, the matter may be settled directly by negotiation between the owner and contractor, and if this fails, the DRB mediates the dispute and recommends a settlement.

Over 20 years of experience with the DRB process have now been accumulated. The reduction in the cost and time required for dispute resolution has been so substantial that the process has spread from tunnel and underground construction to general heavy construction and even to complex commercial building projects. The scope of DRB activities has similarly been extended from its original focus on differing site conditions to include all forms of technical and contractual disputes between the owner and contractor.

A comprehensive discussion of the history, operation, and effects of the DRB process is given in the "Construction Disputes Review Board Manual" by Matyas, Mathews, Smith and Sperry, 1995 (McGraw Hill).

Beyond improvements in the contract form, an owner contemplating an urban underground construction project can greatly reduce the risks of delay and dispute by undertaking an active precontract construction planning program. Such a program might include the following components:

1. Site clearance—advanced utility relocation, demolition, and underpinning, performed during completion of final designs, to permit main contracts to proceed more promptly upon award.

2. Access and working area restrictions—advance negotiation of regulations for traffic maintenance and for public and private access through the construction site.

3. Right-of-way—in addition to R.O.W. for permanent construction, negotiate in advance access to and use of temporary construction areas, both the contractor working and storage areas and for temporary construction such as underpinning.

4. Disposal areas—although it is unwise to mandate a specific disposal area, the owner can take a strong lead in negotiating for possible areas that will be made available to contractors. This matter has become increasingly critical with the growth of environmental impact regulations. It is now virtually mandatory for the owner to identify at least one approved disposal site if he expects to get any bids.

5. Long-lead items—advance procurement by the owner of permanent materials and operating equipment that are likely to become construction bottlenecks if left to the contractors.

An important aspect of a precontract construction planning program is that this work can generally be done better by the owner than by the contractor, because the owner has more time at his disposal, and because he probably has a better negotiating position with respect to many government regulatory agencies. owners who try to allocate risk of negotiating all construction permits to the contractor merely convert this risk into the certainty that the negotiations will be more hurried, less effective, and more costly than if they themselves had done the work (to the extent they could) before calling for bids.

One of the most important risks facing a contractor, particularly in urban construction projects, is that of delays caused by third parties not subject to control of either the owner or the contractor. The standard contract clause provides for an equitable adjustment to time of completion for delays outside of the contractor's control, but no increase in compensation unless there is a physical change in the work. This simplistic legal view ignores the costs of the contractor's supervisory staff and other overhead accounts. Bidders do not ignore these matters, and appraisal of the potential for third party delays is an important component of the contingency included in the bids when they are finally assembled.

Some forms of third part delay, such as environmental disputes, have become so virulent as to preclude advertising for construction until they are resolved. However, even after the contending parties have buried the hatchet, the risk of continuing environmental and other third party squabbles is not unappreciated by contractors. Apart from jawboning, the owner has little direct influence on this risk. However, he may assume part of it, by providing payment for standby time and overhead costs attributable to third party delays. The owner here trades the risk of incurring standby costs against the reduction of the risk of high bid contingencies.

If owners are moving (as they must, either in enlightened self-interest or under the prodding of the courts) away from allocating all risks to the construction contractor, it is evident that more detailed engineering and construction planning are required to define the new allocation of risks. The quality of such services is elusive, but is unlikely to be in inverse proportion to their cost. owners who have taken to securing their engineering services on the basis of the lowest price offered in competition are unlikely to secure the most thorough preconstruction engineering services. They thus allocate to themselves the risk that problems that might have been uncovered and provided for beforehand will crop up only during construction, when the resulting costs will be much higher, although less visible to the auditors.

Construction is a highly complex business. Guidelines, recommendations, contracts, even legal rulings, can only provide direction for judging particular situations. Among the most important and most difficult to define factors in evaluating and allocating risk are the reputations of the parties to the contract. Some owners (and some engineers) have earned such reputations that reputable contractors will not bid on their projects; others have reputations that attract bidders who would pass up such work in other jurisdictions. Conversely, some contractors have earned reputations that invite contract administration "by the book," while others enjoy the ability to secure many contract modifications by negotiation. The risk of an unfavorable reputation (or the benefit of a favorable one) is earned by all parties over a long period, is not allocable, and is not rapidly changed.

Underground construction is always an adventure, and will always have risks. But these risks need not be unmanageable. With experience and forethought, it is possible to draw up contract documents that are equitable for both the owner and contractor, and to produce a tunnel project that is a source of pride for all concerned.

NIGHT TIE AND SURFACING WORK

1995 SOUTH FLORIDA RAIL CORRIDOR TIE PROJECT

By: Jerry P. Epting*

In 1993, CSX Transportation met with representatives from the State of Florida, Herzog and Tri-County Commuter Rail Authority to discuss the feasibility of upgrading the track segment between West Palm Beach, FL and Fort Lauderdale, FL, most commonly known as the South Florida Rail Corridor. The project involved the installation of 34,000 crossties over a thirty-seven (37) mile portion of the corridor, twenty-seven (27) miles of out of face surfacing, surfacing fifty-four (54) turnouts and the rehabilitation of ten (10) major road crossings. In this instance, the crossings were upgraded utilizing full depth rubber material. In addition, the project involved shoulder ballast cleaning in the areas worked with our major timbering and surfacing units. Also, due to the close proximity of local residential areas, environmental and other housekeeping challenges were present.

The ability to secure track time within the corridor utilizing traditional daylight hours would have been extremely difficult considering the traffic patterns on this particular segment. The rail corridor has a tonnage rating of 8 MGT per year, with the majority of the traffic related to the transport of rail passengers. The Tri-County Commuter Rail Authority operated thirty (30) commuter trains per day thru the work limits. Amtrak operated five (5) inter-city passenger trains as well. Combined with CSXT's eight (8) locals and five (5) thru freights, the traffic patterns made planning for sufficient track time to complete the project a major focus. In order to maintain the current operating speed of 79 mph and, subsequently, the operating schedules for passenger and commuter trains on this corridor it was imperative that all entities approach the project with the mentality that the work must be completed. The negotiations that would follow certainly addressed the question of how the work could be accomplished with as little disruption as possible to the existing passenger and commuter train schedules. Also at stake were issues related to the cost effective installation of the required materials, especially considering the limited track time available. Considering the above factors, the decision was made to perform the required work at night. With this decision came concerns centered around the safety and quality of the work that could be performed. CSX had very limited experience in operating major production forces outside traditional daylight windows. However, to accomplish the desired results, night time work seemed inevitable. This brought about a term that I am sure we have all grown accustomed to within the rail industry and that is "CHANGE." At CSX, we have provided training to our managers to help them successfully manage employees through the process of change. With any major change, we see a variety of human emotions that naturally tend to resist the element of change. Considering the magnitude of the change brought about by the night time work, I knew that in order to successfully lead this process, I must first get myself thru the "CHANGE MODEL." In accomplishing this task, the first concern was to establish procedures that would insure the safety of operations. Of prime importance was acquiring sufficient lighting equipment to insure that all affected employees would have clear visibility. We had to develop equipment consists that would maximize our lighting resources. We next had to consider the employee who may have had to work alone or without the benefit of our primary lighting sources. In order to address this issue, we equipped our employees with lighting units that could be mounted to a hard hat to provide a safe level of lighting. We also equipped our road crossing teams and other support units with mobile light sources that could be transported as they moved between locations throughout the project. With the primary light sources in place, we next began a review of our equipment to insure that we had sufficient lighting for our equipment operators so that optimum efficiency could be maintained. In many

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cases, we added lighting to the equipment, primarily with our spike pullers, tie removal and insertion equipment and spike drivers. As the project unfolded, it was very apparent that the time spent planning our lighting arrangement was time well spent, as this one component certainly affected the safety and quality of the entire operation.

While we were busy working out our plan to perform a safe, quality and cost effective operation, our labor relations personnel were discussing contractual arrangements with the various maintenance-of-way organizations. Also affected were our train control employees and work equipment personnel. Having started this process early allowed us the opportunity to openly discuss and address the many concerns that the various labor organizations had in accepting this magnitude of change. Thanks go out to all who participated in this process. I must state that many of our employees regarded this project as a challenge to their technical and professional abilities and were ready to perform to their maximum potential. Good planning and the attitude among our employees certainly lead to the positive results that we experienced on this particular project.

While we were planning our lighting needs and negotiating labor matters, our local forces were involved in unloading crossties and ballast. In order to accomplish this phase of the project we had to unload on the weekends. The weekends provided a more favorable time frame with which to unload due to a decrease in commuter train traffic. Considering the time frame, we had to install the ties, it was imperative that our materials be unloaded properly. We couldn't afford to move crossties ahead to fill voids and still consistently meet our required productivity rates. The same set of circumstances existed with respect to the way our ballast was unloaded. Pre-planning trips made by local forces identified areas where ballast sections were lean and other areas where extra ballast may be required, such as at turnout locations. With this plan in hand, our forces were able to unload all materials to match existing field conditions. As the work progressed, it was evident that our unloading forces had met their responsibilities.

With our lighting arrangements having been completed and our materials on the ground, we then began to examine the actual window that we could utilize to accomplish a safe and cost effective installation. In addition to the commuter train and passenger train traffic, we also had to consider freight traffic that must operate in order to meet customer requirements. A window was established to operate all freight traffic between 2000 hours and 0430 hours the next morning. With assistance from our transportation teammates, we were able to expedite our freight traffic through the affected area by 2130 hours each evening. In effect, we had approximately seven (7) hours of uninterrupted track time. This amount of track time was available to us on a five (5) day basis. Our teams utilized a four (4) day work week (Monday–Thursday) with the fifth day (Friday) being utilized for cleanup operations. Our objective was to clear the track each morning by 0500 hours since we would have a Tri-Rail Commuter train ready to run at that time. Of course, we had to involve our train control personnel to insure that the trains received clear signals upon our returning the railroad back to service each morning.

With all major components of our planning completed, on January 2, 1995 we began two (2) days of safety training for all personnel associated with the project. This training allowed all concerned to ask questions concerning the scope of the project. This process allowed us to lead the change in a most positive manner. After completion of the safety training, we separated the two (2) major tie teams involved and the CAT 09-32 tamper and support equipment. Our planning initiatives had allowed us to establish the consists of the teams in the appropriate order that we could begin work immediately without having to switch out various pieces of our equipment fleet. On January 4, 1995, we began tie installation at Fort Lauderdale, FL and Delray Beach, FL, each team consisting of fifty-eight (58) men. The two (2) tie teams worked towards each other with a planned meet to take place at Deerfield Beach, FL, should each be operating on schedule. The CAT 09-32, which was a twenty-one (21) man force, began work at West Palm Beach and progressed in a northerly direction. In addition to the tie teams and CAT 09-32 unit, we also ran two (2) additional switch tampers to facilitate surfacing the forty-nine (49) turnouts located within the confines of the timbering and surfacing work. Also, we ran a shoulder ballast cleaner to improve the overall quality of the work performed and to insure long lasting results. In effect, we were able to clean approximately thirty-seven (37) miles in conjunction with our timbering and surfacing operations. All total, the operation involved over 140 employees in various assignments.

Due to the large area covered by the various forces, we established a command post at Fort Lauderdale, FL. This post was covered by a project coordinator whose responsibilities involved receiving and distributing information between our Jacksonville, FL offices and those employees actually involved in the tie and surfacing installation. Any changes that would be required could easily be communicated to field personnel and any problems encountered in the field requiring system support could be readily addressed. Effective communication and teamwork were instrumental in completing the project on time.

As the men began to adjust to the change in work hours, our productivity continued to improve. The tie teams started the project by installing 1,100 plus ties per team during the first week. By the second week of operation we were successfully installing over 2,000 ties per night with each team. Considering the apprehension surrounding this project, we knew that with these results we had successfully managed to lead our employees through the change model. With these types of results, we were able to complete the timbering portion of our project two (2) days early, moving on to the next location. Our surfacing efforts drew to a conclusion on January 27, 1995, the original scheduled date for completion of all proposed work.

Overall, the project was a success for CSX Transportation, the State of Florida and Tri-County Commuter Rail Service. As you will recall, upon learning of the proposed night time work schedule, our first concern was the safety of operations. We were able to accomplish the completion of the project without any personal injuries. This was certainly a tribute to the employees involved throughout the entire process. We also learned that with the safety, quality, and productivity levels we achieved on this project, we could apply this concept to other parts of our property and feel confident in our ability to perform successfully. This was yet another part of our efforts to transform CSX into a high performance company.

HIGH ADHESION LOCOMOTIVE THERMAL-MECHANICAL RAIL SURFACE LOADING

By: G. J. Moyar* and D. H. Stone**

Introduction

While the new high adhesion locomotives are performing well, it may be appropriate, at this early stage of associated rail maintenance experience, to consider some of the theoretical increases of rail surface loading anticipated with these driving wheels in order to help prepare rail maintenance plans. A recently developed simplified thermo-plasticity tread surface computer model for estimating the resulting stresses and plastic strains in the wheel treads of high adhesion locomotives is summarized. The nature and severity of the thermo-mechanical (TM) tread surface material cyclic loading associated with this new generation of driving wheels is then illustrated for a particular high traction AC locomotive starting operation (45% adhesion @ 10% slip) vs. a typical DC operation (35% adhesion @ 5% slip). A maximum flash temperature rise of 372°C (670°F) is predicted for the AC locomotive vs. 263°C (473°F) for the DC locomotive. The maximum effective TM stress is 33% lower, the residual tensile stress is 86% lower, and the plastic shear strain increment is 66% lower with the DC locomotive.

It may be that the high surface tractions achievable by high power AC driven locomotives will produce a type of rail defect, known in Europe as the "squat," to North American experience. While the thrust of this paper is based on a theoretical analysis, it may be well for track engineers to be on the lookout for this type of defect.

The Thermo-Mechanical Model

What is the nature and severity of the TM tread and rail surface material loading associated with this new generation of locomotives, and what surface failure mechanisms may be anticipated? A preliminary attempt to explore this conceptually and theoretically was made by the authors in a recent paper (1). In that paper we concentrated on potential wheel damage; while the important companion issue of rail damage is taken up here.

It has long been recognized that even moderate amounts of sliding during rolling can cause transient or flash surface temperatures that, although shallow, can contribute significantly to eventual surface breakdown. Kelley (2) explained in his study of gear surface failure that when the surface temperature of the material is abruptly raised relative to the adjacent subsurface, the surface expands against the restraining subsurface material producing in-surface compressive stresses that can significantly modify the multiaxial Hertzian stress state. In the case of gears, the mating tooth surface having the slower relative speed (analogous to the rail in contact with a driving locomotive wheel) was found to have the higher range of shear stresses and be more susceptible to surface fatigue.

Such transient temperature "spikes" have also been calculated for railroad wheels and rails under combined rolling and sliding (3). The steep subsurface gradients usually associated with transient temperatures may also be calculated (4). At sufficiently high slip and wheel loads such flash temperatures can produce undesirable material transformations in the steel wheel tread or rail. The present authors (5) have applied such thermal analyses to the evaluation of the reduction in surface fatigue life of braked railroad wheels. The TM model developed in our recent paper (1) was part of an initial attempt to extend such an approach to a consideration of the effect of operating conditions on driving locomotive wheel treads.

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As the details of the model have been presented elsewhere (1) only the details of the transient temperature analysis will be presented here. When wheel motion departs from pure rolling, the resulting rail friction leads to transient surface temperature peaks or spikes within the contact area whose magnitudes depend on the available friction coefficient, contact pressure, speed and amount of slip or creep. The resulting temperatures have been called "flash" temperatures, since their peaks are very brief and they do not penetrate deeply into the tread. Nevertheless, they can contribute significantly to the severity of the TM loading of the surface layers of both the wheel and rail. The form of the solution for maximum temperature rise, •m, of a driving locomotive wheel from Tanvir (3) is

$$\theta_m = \frac{C}{K} p\mu \left(\frac{asV}{\pi}\right)^{1/2} [(1+r)^{1/2} - 1] \tag{1}$$

Here K is the thermal conductivity, p is the maximum Hertz Pressure, μ is the friction coefficient, a is the contact patch half-width, s is the thermal diffusivity, V is train velocity and r is the wheel slip ratio. We obtain a value of C equal to 2.20 for this situation. A numerical example of this type of transient thermal analysis is provided in Figure 1, which shows the variation in flash temperature through the contact patch of width 2a, for the particular base case locomotive wheel operation. The peak value given by Eq. 1 is reached after the center of contact but before the trailing edge of contact is reached. The associated thermal strain is directly proportional to this temperature.

It must be recognized that surface thermo-plastic strain and stress pulses, such as those simulated above, occur in the thin transient "thermal skin" that rides on the body of a steadily changing wheel tread undergoing longer range thermal and contact stresses. These subsurface conditions determine the mean stresses upon which the transient surface stress-strain cycle is superimposed. Thus the use of the phrase "residual stress," following a single rolling cycle at the surface, cannot be taken as an absolute value without some consideration of the hot mean or current ambient tread state which must also reflect the concurrent subsurface condition and its TM history. While such considerations are beyond the scope of the present paper, the cyclic surface strain and stress *ranges* estimated in our simple thermo-plastic contact model are believed to provide a good estimate of the relative severity of locomotive tread operating conditions.

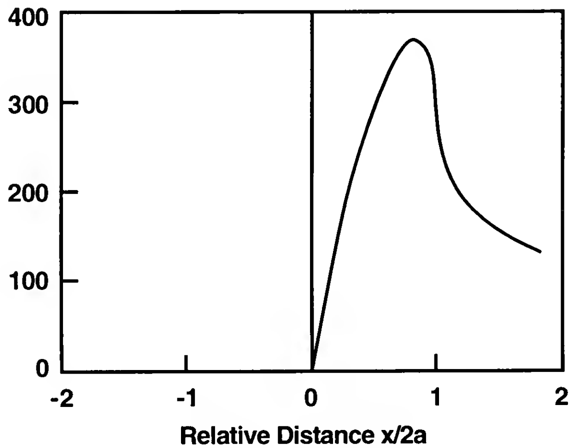

Figure 1. Example Surface Temperature Spike.

It should also be recognized that the simple total deformation plasticity used for illustration here is not entirely adequate for a realistic treatment of the plastic strains and residual stresses that occur in non-proportional loading. This is a subject for future work.

Squat Defects and Their Origins

Squat defects first appeared in the early 1970s on British Rails London-Bristol mainline shortly after the introduction of their high speed passenger trains, and appeared on other lines after introduction of high speed service. Squats also began to appear on the JNR Shinkansen lines. Extensive investigations at British Rail Research came to the following:

> "In attempts to determine the initiating mechanisms of this defect, it has not been possible to correlate the occurrence with any one specific track, or traffic parameter, or any metallurgical feature of the affected rails. It has only been possible to state that the defects were found predominately in high speed/high tonnage lines. However it was observed that approximately 75% of the defects were associated with either periodic indentations in the rail head, corrugations or welds. This suggested that the defect could result from excessive plastic deformation due to the higher stress conditions existing at these features. At this stage it was concluded that the defect was a rolling contact fatigue problem. . . . (6)

The authors theorize that the increased locomotive velocities had the effect of increasing the flash temperature at the wheel rail interface and thus the contact stresses. In North American operations, the magnitude of the friction coefficient and the slip ratio will be increased by the use of AC locomotives at starting and low speeds which, referring to Equation 1, would also have the effect of increasing the flash temperature.

Squat defects appear as a black spot on the running surface of the rail as shown in Figure 2. The spot is the result of a relatively shallow crack that starts at the rail surface and turns into a plane parallel to the running surface. Frequently a branch crack forms a transverse defect as shown in Figure 3. These defects also cause problems during ultrasonic rail inspection due to the fact that the crack parallel to the surface can mask the more dangerous transverse branch.

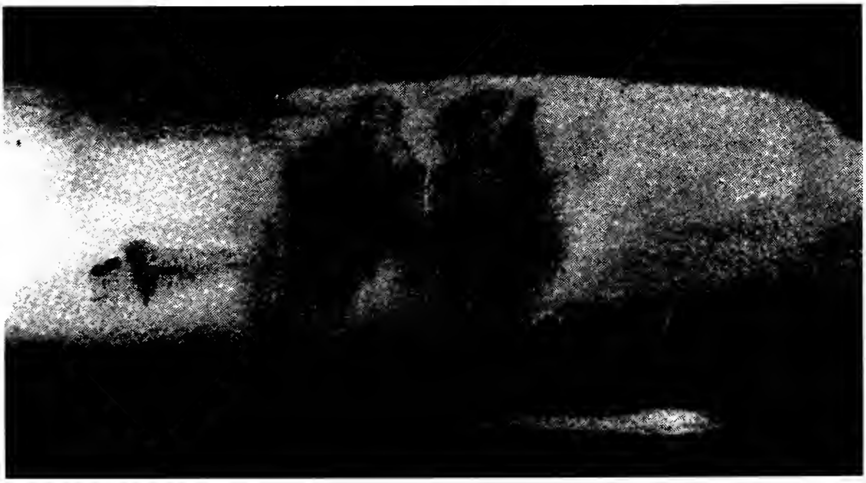

Figure 2. Appearance of Squat Defect on Rail Running Surface (6).

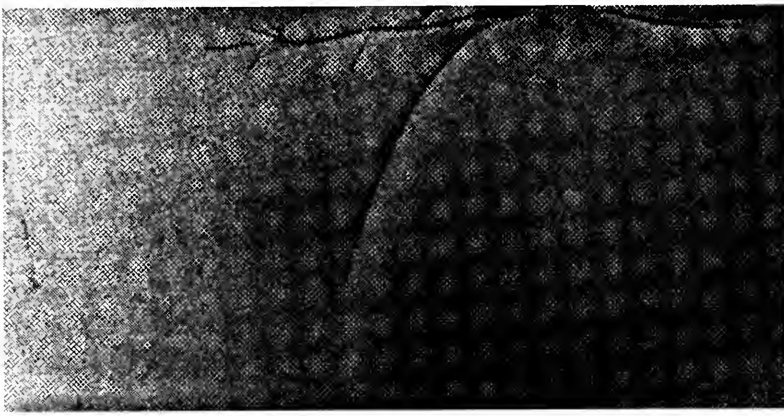

Figure 3. Cross Section of Squat Defect in Rail Head (6).

Severity Index

In the course of this comparison of severity of loading, a Thermo-Mechanical Severity Index (TMSI) was developed based on the expression for peak flash temperature and non-dimensional parameters that characterize the plastic potential of the Hertzian traction and the thermal shock level:

$$TMSI \ \frac{p}{k} \ \mu L^{1/2}(1 \ r)^{1/2} \ 1 \qquad (2)$$

Where p/k is the usual ratio of maximum Hertz pressure to yield in shear (at the maximum temperature reached), L is the Peclet number $[Va/(2s)]$, μ is the adhesion or rail friction coefficient and r is the wheel slip ratio.

It appears that a quick initial estimate of the severity of locomotive operation may be obtained by calculating this non-dimensional grouping for the particular operation and using charts to estimate the relative residual stress or plastic shear strain, etc. The value of TMSI for the AC base case is 3.628.

For comparison to these base case results, a less severe TM loading case with the same wheel and vertical load is selected. This reduced severity case is intended to represent conventional locomotive starting conditions and is designated, "DC." A direct comparison of traction conditions and predicted results, including flash temperature, the TMSI, stresses and plastic surface shear strain is provided in Table 1. Note that a 50% reduction in slip and a 22% reduction in adhesion leads to a 61% reduction in flash temperature, a 31% reduction in maximum effective stress, an 86% decrease in residual tensile stress, and a 66% reduction in surface plastic shear strain.

Existing Conditions that May Preclude Squat Formation

One of the most important differences between European and American conditions is the use of higher strength rail in North American track. The standard British rail steel, BS11, has a minimum yield strength of approximately 65,000 psi, while 300 minimum Brinell AREA rails have a minimum tensile strength of approximately 103,000 psi which provides a 60 percent additional resistance to plastic flow.

Table 1.
Comparison of AC to DC Tread TM Starting
Loading Severity for 154 kN (34,614 lbs.)
on 1.016 m (40") wheel @ 3.1 m/s (7 mph) speed

	AC	DC	% HST
Adhesion %	45	35	25.0
Slip %	10	5	.5
Hertz Pressure	114	114	982.0
(MPa)	2	2	
Flash Temp. Rise (•C)	372	146	351.0
TMSI	3.6	1.1	–67.6
Max. Eff. Stress	119	798	–32.9
(MPa)	0		
Max. Sxx Range	243	184	–24.1
(MPa)	6	8	
Max. Shear Stress (MPa)	648	446	–31.2
Max. Resid. Stress (MPa)	505	70	–86.1
Plast. Shear	0.9	0.3	–65.9
Strain (%)	11	11	

Another important consideration is that high speed European passenger locomotives operate nearly continuously at speed, while a North American freight locomotive operates at maximum adhesion and slip at starting and possibly during the ascent of heavy grades. Thus the European locomotive may be imparting damage over the majority of track while the North American locomotive may be imparting maximum damage at relatively few track locations.

Finally, it should be emphasized once again that this is a theoretical analysis which contains assumption and limitations. It must be recognized that surface thermo-plastic strain and stress pulses, such as those simulated above, occur in the thin transient "thermal skin" that rides on the body of a steadily changing wheel tread undergoing longer range thermal and contact stresses. These subsurface conditions determine the mean stresses upon which the transient surface stress-strain cycle is superimposed. Thus the use of the phrase "residual stress," following a single rolling cycle at the surface, cannot be taken as an absolute value without some consideration of the hot mean or current ambient tread state which must also reflect the concurrent subsurface condition and its TM history. While such considerations are beyond the scope of the present paper, the cyclic surface strain and stress *ranges* estimated in our simple thermo-plastic contact model are believed to provide a good estimate of the relative severity of locomotive tread operating conditions.

It should also be recognized that the simple total deformation plasticity used for illustration here is not entirely adequate for a realistic treatment of the plastic strains and residual stresses that occur in non-proportional loading. This is a subject for future work.

Discussion

Will squat defects appear in North America? It is probably too early to tell. However, it may be prudent to watch for the appearance of black spots near unit train loading facilities and on heavy grades where AC traction is employed.

On the other hand, Swenson (7) has reported minimal wheel wear on AC locomotives in service on the Burlington Northern. While the use of radial trucks on these locomotives is undoubtably a major contributing factor to minimizing wear and shelling behavior, the excellent performance of these wheels may indicate that these locomotives spend only a minor portion of their service at maximum slip and adhesion, and sufficient high thermo-mechanical stress cycles with not be induced in the rail head to cause the formation of squat defects.

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FATIGUE TESTS OF RIVETED BRIDGE TENSION MEMBERS

By: Jeffrey D. DiBattista, Graduate Student*

The remaining life of existing bridges is a topic of increasing importance for the railway industry in today's competitive economic environment, and fatigue failure is a primary factor that must be considered in the evaluation of older bridges. Because most older bridges were built with riveted rather than bolted or welded connections, the fatigue strength of riveted connections is of major importance. The study reported herein describes the physical testing of bridge members taken from an existing structure in order to determine their fatigue strength. The results supplement existing knowledge and show that Category D most appropriately describes the fatigue strength of riveted connections. A practical and effective repair technique was also developed and tested.

Introduction

From the middle of the nineteenth century until approximately 1960, most wrought iron and steel railway bridges were constructed using rivets as the structural fastener. Many of these old bridges have endured the passage of thousands of trains, and economic factors often make it essential that they continue to function for many years to come. The recurring application of load on these structures can lead to fatigue cracking, and possibly even structural failure. These fatigue cracks can occur at much lower loads than those that would normally cause failure.

Fatigue cracking of bridges can also be accelerated by corrosion, increasing volumes of traffic, and vehicles heavier than those originally anticipated. Consequently, many riveted structures are approaching or have reached the end of their theoretical fatigue design lives, requiring their replacement or renovation. This is most significant for railway bridges, which are generally subject to higher live loads than are highway bridges. It would be prohibitively expensive to attempt to replace or renovate all of these old bridges, as they number in the thousands in North America alone. Bridge owners, therefore, look to research for information enabling more accurate predictions of the remaining fatigue lives of their structures so that premature bridge repairs or replacements can be avoided, while at the same time maintaining adequate margins of safety.

Despite the large number of riveted bridges still in service, only limited research has been performed on riveted connections as compared to research on welded or bolted connections. Until recently, the absence of research can be attributed to a lack of appreciation for fatigue as a failure mode in the design of civil engineering structures. Only seven major research programs were identified in a search for literature on tests of full-scale riveted connections.

It is important to appreciate the parameters that affect the initiation and growth of fatigue cracks. A fatigue crack usually originates at a stress concentration, such as an existing flaw or an abrupt geometrical discontinuity in the material. The dominant variables that influence the fatigue strength of structural steel are the net section stress range, the number of cycles of applied stress, and the type of structural detail (1).

Net section stress range is defined as the algebraic difference between the maximum and minimum stress acting on the net area of the cross-section of a detail. (Net area is the value that is obtained when the area of geometrical discontinuities, such as rivet holes, are deducted from the gross area of the cross-section.) As net section stress range increases, the number of cycles of load that may be applied decreases. This relationship is linear when each of stress range and number of cycles is expressed logarithmically, and the slope of this line is generally –1/3 (this slope is often more simply

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referred to as three in common practice). This relationship holds true until a certain stress range is reached, below which there is no fatigue crack growth. Thus, there is also a horizontal portion in a plot of log stress range versus log number of cycles, known as the *constant amplitude fatigue limit*, below which there is no fatigue crack growth.

Various types of connections have different fatigue strengths. In general, bolted connections have the greatest fatigue strength, welded connections have the least fatigue strength, and riveted connections lie in between. These various structural details are categorized alphabetically in North American practice (2, 3, 4): Category A has the greatest fatigue strength and Category F has the lowest fatigue strength. The behavior of riveted connections is generally considered to be best represented by Category D.

Experimental Program

Fatigue tests were conducted on tension members taken from a riveted railway bridge that was dismantled in 1991 after 80 years in service (5). The bridge was located over the Miette River, west of Jasper, Alberta, and it was part of the Canadian National Railways (CN Rail) mainline system. It was a straight, single track, steel five-panel through-truss structure with a single span of 125 feet. The bridge had a width of 18 feet between truss centerlines, and the floor system consisted of transverse floorbeams and longitudinal stringers. All connections for both truss and floor members were riveted, and all primary structural members were built-up riveted sections. The track was centered along the longitudinal axis of the bridge and was not ballasted, that is, the timber sleepers rested directly on the stringers. A photograph of the bridge in service is shown in Figure 1.

Of specific interest are the four primary tension diagonals that were tested as part of this study. The diagonals were riveted built-up sections that used 6 in. x 3.5 in x 7/16 in. angles and a 14 in. x 3/8 in. web plate. The short legs of the angles were joined to opposite sides of the web with rivets 7/8 in. in diameter so as to form an I-shape. Two gusset plates, each 1/2 in. thick, connected each end of the diagonals to other members at the top and bottom panel points. Each gusset plate was attached to the diagonal with twenty-six 7/8 in. diameter rivets. Before the bridge was dismantled, service load strain

Figure 1.

measurements were taken on one tension diagonal and on one stringer. (The stringers were later tested by Adamson [6].) The strain measurements showed that the stress range in the members was very low, indicating that the results from the fatigue tests likely would not be affected by prior fatigue damage.

When the diagonals were taken from the bridge, the gusset plates at the panel points and the diagonal were removed as a unit. Each diagonal was then cut transversely in half in order to obtain manageable lengths, and this gave a total of eight specimens. Four of these had a portion of diagonal attached to an upper panel point, and the other four pieces had a portion of diagonal attached to a lower panel point. One specimen was damaged after removal from the bridge and was not tested.

In order to test the seven remaining specimens in such a way as to model service conditions as accurately as possible, it was first necessary to determine the location of the *critical detail*. The critical detail is the location in the specimen at which the first crack is expected to appear, that is, the cross-section where the largest stress range is present. The geometry of the specimens taken from the bridge was such that the critical detail was in the diagonal at its connection to the gusset plate. Consequently, when the specimens were prepared for the tests, an adequate area of gusset plate was kept to ensure that the first crack would form in the diagonal, that is, the same failure mechanism that would have occurred in the original structure. All specimens were also completely sandblasted so as to permit improved crack detection.

Each specimen was loaded in uniaxial tension fatigue using a steel loading frame and a single high-speed hydraulic jack. Because of the large cross-sectional area of the full-scale specimen, large forces, in the order of 200 kips, were required to achieve the desired stress range at the critical detail. In order to exert these forces on the specimen, a lever system providing mechanical advantage to the jack was incorporated into the load frame. The lever transferred load through a spherical roller bearing, a large-diameter steel pin, and finally into a load transfer plate that was attached to one end of the specimen with a pretensioned bolted connection. The other end of the specimen was attached to a support in a similar fashion. This system ensured that only uniaxial force was applied, because the bearings did not allow moment resistance to be developed at the end connections. Cyclic loads were provided by an electronically-controlled servovalve that regulated the hydraulic jack to provide the desired force. The stress range in each specimen was controlled and monitored using a computer data acquisition and control system.

The literature review indicated that very few tests have been conducted at stress ranges below 11 ksi, and no tests of full-scale riveted tension members have been conducted below 18.1 ksi. This is significant because most bridges in service rarely experience stress ranges above 10 ksi, thus many of the tests performed to date do not reflect the conditions that bridges endure in reality. As a result, the tests reported herein were carried out at comparatively low critical net-section stress ranges—two specimens at 10.6 ksi and one each at 10.4 ksi, 10.1 ksi, 10.0 ksi, 9.6 ksi, and 9.4 ksi. In order to avoid instability in the load frame, the minimum net section applied stress for all specimens was approximately 1.5 ksi of tension. Net cross-sectional areas, and thus stress ranges, were calculated using the commonly known $s^2/4g$ rule (7), although this method has limitations when applied to fatigue loading (5).

It was also necessary to define a failure criterion that closely approximated failure of the member, that is, its inability to carry the applied load. The failure criterion selected was the severing of one element of the built-up cross-section of the diagonal and the detection of a crack in a second element. This definition of failure was used to maintain consistency with similar tests of flexural members performed by others (6). In order to avoid damaging the loading system, it is usually necessary to stop a test before the member is no longer able to carry the applied load.

Results

In general, the behavior of the specimens was similar in all cases. A crack formed at the critical detail, that is, the hole for a gusset-plate-to-diagonal rivet, and this crack eventually severed the angle. After some time, a second crack started in another element of the diagonal, either another angle or in the web. The test was then stopped and the total number of cycles required for the crack to begin in the second element was recorded. The number of cycles required until failure ranged from approximately 1.5 million to 5 million cycles. Figure 2 shows a typical fatigue crack in the diagonal, and the results from the tests are illustrated in Figure 3. The AREA Category D and Category C fatigue design curves as well as fatigue data from other full-scale tests of riveted connections are also shown in Figure 3. It can be seen that the results from these tests suggest that the fatigue strength of this type of connection is best represented by Category D.

One test was allowed to continue until the specimen was no longer able to carry the applied load. This was done in order to determine how closely the failure criterion as defined in the study approximated actual failure, that is, the inability of the specimen to carry the applied load. It was found that only a very small number of cycles was required between the time that the failure criterion was met and the time that the specimen could no longer carry the applied load. This indicates that the failure criterion was appropriate.

In another test, the specimen was repaired after an element had severed and the test was then restarted. This was done in order to evaluate the effectiveness of the repair technique. Splice plates were used in this repair to carry the load around the cracked location. The plates had dimensions of 1 in. x 13 in. x 44 in., and were designed so that the net section stress range in the plates would not exceed the AREA Category D constant amplitude fatigue limit of 7 ksi (based on the conservative assumption that the uncracked portion of the specimen carried no load). Each plate was fastened to the specimen using six pretensioned 7/8 in. diameter A325 bolts at each end.

The results from the repaired specimen show that the repair technique was a success. No new cracks were discovered after the repair was made and the existing crack did not propagate further, even though many additional cycles of stress were applied. It may be concluded, therefore, that a repair of this nature could be used as either a temporary or permanent solution in actual applications, although more tests should be conducted to validate the results from this one test.

Conclusions and Recommendations

Measured strains in the diagonal did not indicate that any fatigue damage existed when the tests began, and thus the test results should accurately reflect the fatigue strength of the members. These results

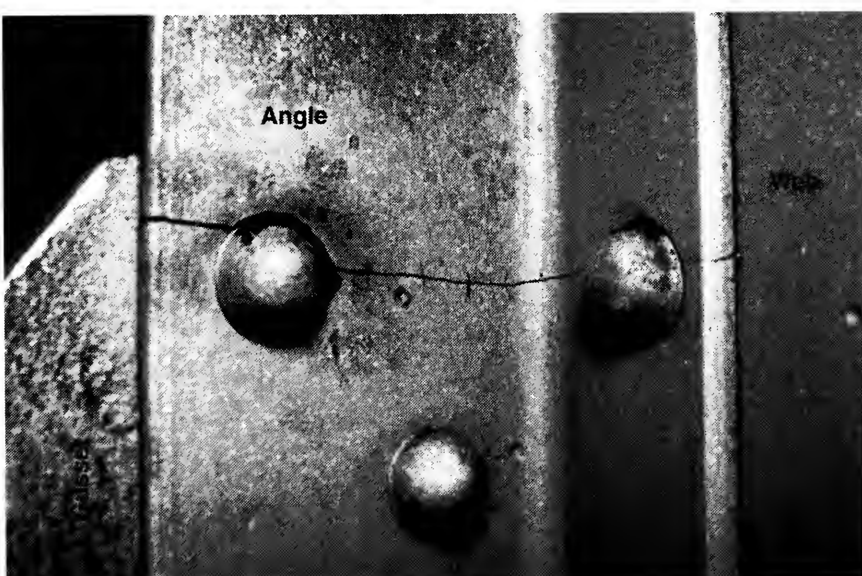

Figure 2.

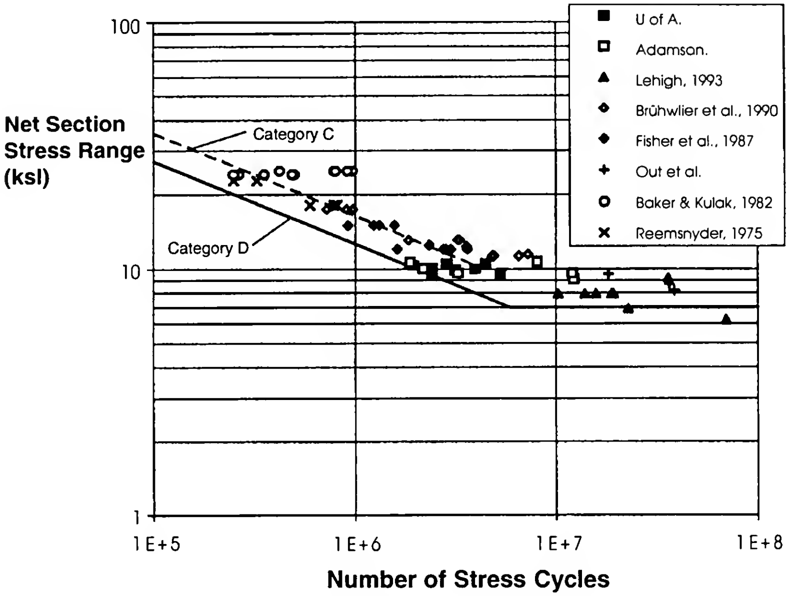

Figure 3.

show that the fatigue resistance of the diagonals at their connection to the panel points slightly exceeded the Category D fatigue strength curve of the American Railway Engineering Association standard.

The failure criterion selected, in which one element is severed and a crack has appeared in a second element, closely approximated an inability of the specimen to carry the applied load. The repair of a cracked tension member to gusset plate connection with pre-tensioned bolted splice plates extended the life of the connection significantly.

Further testing is required in order to define the constant amplitude fatigue limit for riveted connections. Preferably, these tests should be conducted on connections in which the rivets are in bearing and where the applied stress ranges are below about 10 ksi.

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PUBLISHED AS INFORMATION
COMMITTEE 1—ROADWAY AND BALLAST

Chairman: J. R. Zimmerman

Assignment D-2-3-89: Mill Abrasion Test Study

By: J. K. Lynch*

Executive Summary

Sub-Committee No. 2—Ballast conducted an extensive study of the Mill Abrasion test to determine the repeatability and reproducibility of the test method conducted by seven independent laboratories on identical ballast materials.

Test results showed considerable variability between the participating laboratories. Accordingly, an audit was conducted of the seven participating laboratories to determine that the correct test procedure was used and to verify the accuracy of the test results. The results of the audit confirmed that each of the laboratories used correct laboratory procedures and generated correct test results in their identical tests.

The primary goal of the Subcommittee was to incorporate the Mill Abrasion test in the AREA Ballast Specification, provided the test repeatability and reproducibility were satisfactory to consider the Mill Abrasion test a viable physical test for ballast materials.

Additional research is needed to verify results.

The Development and Study of the Mill Abrasion Test

Goal

The goal of this project was two-fold: 1) determine the repeatability (replication of test results within the same lab) and reproducibility (replication of results between different labs) of the Mill Abrasion test method and 2) if the repeatability and reproducibility were satisfactory, the committee would then consider incorporating the Mill Abrasion test in the AREA Ballast Specification.

What is Mill Abrasion

The Mill Abrasion test is conducted in the following manner. A sample of ballast weighing approximately 7 pounds (3 kg), and consisting of rocks ranging in size from 1 1/2" to 3/4", is placed in a porcelain jar. The jar is then filled with approximately 1 gallon (3 l) of water, sealed, then rotated on jar mill rollers for 10,000 revolutions at a rate of 33 RPM (approximately 5 hours). After rotation, the sample of abraded ballast is washed, sieved, then oven dried. The Mill Abrasion result is equivalent to the amount of sample that passes a No. 200 sieve (following abrasion) as a percentage of the original weight of the test sample.

The test is similar in principle to the Los Angeles (LA) Abrasion test with two important differences. Unlike the LA Abrasion test, the Mill Abrasion test does not incorporate the use of steel balls to pound the rock. The destructive forces to the ballast are instead provided by the rocks themselves which tumble and rub against each other while the jar rotates. The second difference is the container is filled with water during the Mill Abrasion test whereas in the LA Abrasion test, the container is left dry.

It is important to understand that the LA Abrasion test and the Mill Abrasion test ultimately measure two different properties of the rock. The LA Abrasion test in essence measures how tightly the rocks' minerals are bound together, while the Mill Abrasion test measures the rock's minerals resistance to abrasion.

*Chairman Subcommittee 2—Ballast; Vulcan Materials Co.

Committee Approved Mill Abrasion Test Method

The Mill Abrasion test method used in this study was comparable with test methods used by some railroads and private agencies. Although similar in nature, they contained minor differences. The Ad Hoc committee took these methods as a base and modified their contents into a draft test method leaving the basic structure and contents intact. The method used in this study is outlined below. Note that there is no Mill Abrasion test method in the American Society for Testing and Materials (ASTM).

AREA Committee Approved Mill Abrasion Test Method

1. LABORATORY TESTING APPARATUS
   A. Sieves—A nest of two sieves, the lower being a No. 200 (0.075 mm) sieve and the upper a 3/8 in. (9.5 mm) sieve, both conforming to the requirements of ASTM E-11.

   Note: Recommend use of a 10 in. or 12 in. diameter sieve or comparable size to allow for sufficient sieving capabilities.

   B. Balance—A balance or scale accurate to within 0.1% of test load over the range required for the test.

   C. Container—Porcelain jar of 9 in. (229 mm) external diameter and 1 gallon capacity with water tight cover.

   D. Jar Mill—Mechanical jar mill rubber rollers for rotating the container about its longitudinal axis at 33 RPM (+ or – 1 RPM).

2. SAMPLING AT MATERIAL SOURCE
   A. The field sample is obtained in accordance with current ASTM D75 and reduced to test portion size in accordance with current ASTM C702.

   B. The test portion is then separated into two individual size fractions: (< 1 1/2" to ≥ 1") and (< 1 1/2" to ≥ 3/4"). The separating or sieving procedure is to be performed in accordance with current ASTM 136.

   Note: If testing is to be performed at the material source, GO TO SECTION 3.

   C. Once separated into individual size fractions place:
   - 2000 g (+ or – 100 g) of < 1 1/2" to ≥ 1" material in shipping container with label designating size fraction, material source, and type material.
   - 2000 g (+ or – 100 g) of <1 to ≥ 3/4" material in shipping container with label designating size fraction, material source, and type material.

   Note: The two containers contain enough material to conduct one full Abrasion Test. If more than one test is to be performed, multiply the quantity figures above by the number of tests.

   D. Transport or ship the containers of material to the laboratory performing the Mill Abrasion test(s).

   Note: Care should be taken to package the material to reduce the potential for degradation of the material during transport or shipment.

3. LABORATORY PREPARATION OF TEST SAMPLE
   A. Each size fraction should be thoroughly spray washed and then dried to a constant weight at a temperature of 230 +/– 9°F (110 +/– 5°C).

   B. Hand sieve each size fraction over the sieves listed below to confirm the size of the fraction.

Size Fraction	Sieve Over
< 1 1/2" to ≥ 1"	1"
< 1" to ≥ 3/4"	3/4"

   Note: Hand sieve with agitation sufficient only to ensure that all undersize material passes through the sieve.

   C. Recombine the material to the following test grading.

Sieve Size Passing	(Square Opening) Retained On	Weight of Individual Sizes
1 1/2 in. (37.5 mm)	1 in (25.0 mm)	1500 g +/– 10 g
1 in. (25.0 mm)	3/4 in. (19.0 mm)	1500 g +/– 10 g

TOTAL TEST PORTION = 3000 g +/– 10 g

The weight of the sample prior to the test shall be recorded to the nearest gram. This weight shall be designated (WT) for the calculations.

4. PROCEDURE

A. Sample Preparation

Thoroughly wash and dry the porcelain sample container immediately before inserting the sample.

Carefully place the test sample into the container in a manner that will not cause abrasive action prior to running the test.

Carefully pour 3.0 liters of distilled water into the container. Seal the container and place on the Jar Mill Rollers and rotate the container about its longitudinal axis for 10,000 revolutions at 33 RPM (+/− 1 RPM).

B. Sieving Process

Nest the 3/8 in. (9.5 mm) sieve over top of the No. 200 (0.075 mm) sieve. After rotating the container of material for 10,000 revolutions, carefully pour the contents of the container over the nested sieves. Rinse the container and lid over the nested sieves to ensure all sample particles adhering to the container are returned to the sample.

Thoroughly wash (spray) the material retained on the 3/8 in. (9.5 mm) sieve to ensure all fines have been washed through the sieve. Remove the 3/8 in. (9.5 mm) sieve and carefully place the contents into a drying pan (+ 3/8 in. material).

Thoroughly wash (spray) the material retained on the No. 200 (0.075 mm) sieve to ensure all material finer than the No. 200 (0.075 mm) sieve has been removed. (*Note*: The wash water going through the sieve should be clear.) Carefully place all contents retained on the No. 200 (0.075 mm) sieve into a separate drying pan (−3/8 in. + No. 200 material).

Dry both materials to a constant weight at a temperature of 230 +/− 9°F (110 +/− 5°C).

When dry, hand sieve the + 3/8 in. material through the 3/8 in. (9.5 mm) sieve into a pan. Hand sieve with agitation sufficient only to ensure that all undersize material passes through the sieve. Place the contents that passed through the sieve into the pan of − 3/8 in. + No. 200 material.

Take the material retained on the 3/8 in. (9.5 mm) sieve and weigh to the nearest 1 g. This weight will be designated (W1) for calculations.

Weigh the − 3/8 in. + No. 200 material to the nearest 1 g and designate this weight (W2) for calculations.

5. CALCULATIONS

WT = Total weight of Test Sample prior to running test.

W1 = Weight of material retained on the 3/8 in. (9.5 mm) sieve after test.

W2 = Weight of material passing the 3/8 in. (9.5 mm) sieve and retained on the No. 200 (0.075 mm) sieve after test.

A. Mill abrasion % Loss (MA) = Amount passing the No. 200 (0.075 mm) sieve as a percentage of the original weight of the test sample.

$$MA = [(WT - W1 - W2)/(WT)] \times 100$$

B. Broken material generated (B), % B = [(WT − W1)/(WT)] x 100

C. Proportion of broken material that is finer than No. 200 (0.075 mm) sieve (P),

$$\% P = [(WT - W1 - W2)/(WT - W1)] \times 100 = (MA/B) \times 100$$

6. REPORT

Report MA, B, and P to the nearest 0.1%.

Interlaboratory Test Program

ASTM C 802 Standard Practice for Conducting an Interlaboratory Test Program to Determine the Precision of Test Methods for Construction Materials was used as the guide to develop the Mill Abrasion Interlaboratory Study. The initial goal of the Ad Hoc committee was to obtain ten qualified laboratories to participate in the study. The committee found only seven laboratories to volunteer in this effort. Based on guidance from the ASTM Standard, the committee decided to have each laboratory run five replicate Mill Abrasion tests on each of six homogeneous ballast samples selected for the study.

I. Ballast Sampling Procedure at Quarry Sites

Samples were taken from stockpiles at each quarry in accordance with ASTM D 75 and reduced to test portion size per ASTM C 702. Materials were then separated into two gradations of 1 1/2" to 1" and 1" to 3/4" for shipment. Sample collection, reduction, and shipment was supervised by a representative from the AAR (i.e.: Association of American Railroads).

Samples consisted of 22 pounds of 1 1/2" to 1" material and 22 pounds of 1" to 3/4" material. Two samples from each quarry were sent to each participating laboratory. One sample was used for this study, the second sample was set aside for possible future testing.

II. Laboratory Data

When the sample was received at the laboratory, the two size fractions comprising each sample were combined, resulting in a sample with stone sizes ranging from 1 1/2" to 3/4". The laboratory then performed five separate Mill Abrasion tests on each of six samples resulting in a total of thirty data points. All Mill Abrasion testing was performed in accordance with the Mill Abrasion Test Method. The thirty Mill Abrasion data points generated by each of the seven participating laboratories are as follows.

LABORATORY DATA—MILL ABRASION

	Ballast Material					
LAB	**A**	**B**	**C**	**D**	**E**	**F**
	2.7	3.4	3.8	3.3	7.1	4.4
	2.8	3.3	4.1	3.8	6.4	4.3
1	2.5	3.2	4.4	3.8	6.7	4.4
	3.4	3.0	4.2	3.5	7.2	4.0
	2.6	3.2	4.1	3.8	6.2	4.2
	3.7	3.4	3.9	3.5	6.5	4.4
	2.7	3.4	3.9	3.6	6.5	4.5
2	2.3	3.3	3.6	3.8	6.7	4.4
	2.4	3.2	4.0	3.8	6.6	4.4
	2.4	3.5	3.6	3.8	6.6	3.9
	2.4	3.4	4.3	4.3	6.6	4.1
	3.0	3.8	4.0	3.6	7.1	4.2
3	3.5	4.6	5.1	4.5	7.7	4.5
	3.4	5.0	4.3	3.9	6.5	4.6
	3.5	4.0	4.4	4.9	9.1	5.2
	2.8	3.3	3.7	4.1	6.8	5.0
	2.7	3.3	3.6	3.4	6.9	3.9
4	3.3	3.6	3.6	4.2	6.5	4.4
	3.1	3.4	3.6	3.8	7.4	4.1
	2.7	3.4	3.9	3.8	6.9	4.1
	3.6	4.5	4.4	4.0	7.3	4.3
	3.5	6.0	4.5	4.2	6.9	4.8
5	3.0	5.2	4.2	4.3	7.4	4.1
	3.4	4.9	4.5	4.4	7.5	4.5
	3.4	4.8	4.2	4.2	7.6	4.2
	1.9	2.8	3.3	3.5	5.7	3.6
	2.7	2.9	3.1	3.3	5.9	3.4
6	2.7	3.1	3.6	3.0	6.3	3.9
	2.7	3.9	4.1	3.6	7.0	7.8
	5.9	3.2	3.5	3.0	6.2	3.5
	2.6	3.8	4.5	4.2	6.3	5.2
	3.5	3.5	4.3	4.1	7.1	3.7
7	2.4	4.2	3.6	3.5	6.4	5.0
	2.4	3.2	4.2	4.1	6.8	3.7
	2.1	3.3	3.4	4.0	6.6	4.9

From an inspection of the raw data, it is apparent that the data are not homogeneous. We have performed statistical analyses on this data (not included in this report) that have further confirmed excessive variability in the data. ASTM Standard Practice C 802 suggests poor reproducibility and repeatability of Interlaboratory data may be an indication of problems inherent with the test method. In such cases, it is recommended that precision statements not be developed until a further study of the method is undertaken to determine the cause for the erratic behavior.

Accordingly we elected not to generate precision statements from these Interlaboratory Data.

III. Discussion

As noted above, inspection of the raw Interlaboratory Comparison Mill Abrasion data revealed the data were not homogeneous. Specifically, while data generated *within* a lab was in many cases fairly consistent (i.e., repeatable), there were distinct variances in the Mill Abrasion data generated between the *different* labs for identical materials. For example, for Material B, the average of the five Mill Abrasion tests obtained for each of the seven labs varied greatly, ranging from 3.17 to 5.08.

Further examination of the data showed that some labs consistently averaged higher results than others. It also revealed that Mill Abrasion data generated by some labs for a given ballast material showed greater variability than data generated by other labs for the same ballast material.

When it became apparent that the original data were not homogeneous, Subcommittee 2 performed an audit of the seven participating labs to determine the accuracy of their testing. The following items were the basis of the audit:

- How often the lab ran this test, i.e., routinely or sporadically
- The level of experience the lab technician had running Mill Abrasion tests
- Whether or not the same lab technician generated all of the lab's MA results
- Details about the equipment used by the participating lab to generate their data
- The integrity and condition of the ballast samples that each lab received
- Details about the procedure that each lab used to generate their data, e.g., washing technique, sample preparation and handling, etc.

Each lab was asked if the test method needed to be changed to provide more consistent results.

The audit determined conclusively that the participating labs used the correct laboratory procedures and generated correct results. It also determined there were no problems with the existing test method, nor were changes needed to provide more consistent results.

Conclusions

The purpose of this project was to perform an Interlaboratory Test Program to determine the repeatability and reproducibility of the Mill Abrasion test and analyze the results of the test program. In this report, we have provided results from over 200 tests.

As noted earlier, inspection of the raw data revealed the data were not homogeneous. Accordingly, we conducted an audit to verify that each participating lab used the correct test procedure to determine the validity of the test results. The audit showed conclusively that the participating laboratories used the correct procedures and the validity of the test results was therefore confirmed. Because the data are not homogeneous and based upon guidance from ASTM Standard Practice C 802, the committee did not generate precision statements from the Interlaboratory data.

The committee has provided a valid study with available resources. We can not determine the reasons for the variability in these test results and do not know what significance the Mill Abrasion test might have in determining ballast quality. Additional research is needed to verify results.

MEMOIR
Ben D. Sorrels
1934–1996

Ben D. Sorrels, 61, died in Albuquerque, New Mexico on February 10, 1996.

Mr. Sorrels was born in Borger, Texas on September 3, 1934. He graduated from Texas Tech University in 1958 receiving a degree in Industrial Engineering. He was a member of the R.O.T.C. and Alpha Tau Omega fraternity and was a U.S. Army veteran.

Upon graduation from college, Ben began his career on the Santa Fe Railroad in 1959 at Amarillo, Texas. Over the next 31 years of service on the Santa Fe, he held many engineering and management positions. These included Rodman, Transitman, Assistant Roadmaster, Roadmaster, Assistant Division Engineer, Division Engineer, District Engineer, and his final position of Engineer of Track-Rail in Chicago and Albuquerque. Mr. Sorrels was highly respected by his fellow Santa Fe'ers for his knowledge of track and rail technology.

Mr. Sorrels joined AREA in 1967 and became a life member in 1992. He was a member and past chairman of Committee 4—Rail. Ben was a member of the Roadmasters and Maintenance of Way Association and was its President in 1981.

Ben retired from service with the Santa Fe in 1990 in Albuquerque. Ben is survived by his wife, Leigh, of Albuquerque, four daughters, three grandsons, eight step grandchildren, and his mother, Amorita M. Sorrels, of Guthrie, OK.

Ben Sorrels was highly regarded by the railroad industry for his extensive knowledge of rail technology as well as for his hard work in AREA Committee 4 and the Roadmasters Association. Ben will be remembered for his unselfish work in the industry associations, for his sharp wit, for always having a "new" joke, and for his love for having a good time while reminiscing railroader talk in the evenings. The Santa Fe Railroader's who worked with Ben, his many industry friends across the nation, and his old golfing buddies will miss Ben Sorrels.

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American Railway Engineering Association

BULLETIN

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Front Cover: The Portage Bridge, Portageville, New York. Built 1875. Rising 238 feet above the 70 foot Letchworth Park Middle Falls of the Genesee River.

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Figure 1. On high steel. Inspectors take on the role of a real life "Spider Man."

BRIDGE INSPECTION—MOUNTAINEER STYLE

By: P. M. Roddy*

The Challenge: The safe inspection of a bridge built in 1875, which carries 15 million gross tons annually, is 820 feet long, 236 feet high, and was built at the edge of a 70 foot water fall.

Conrail's tallest bridge is located on the Southern Tier Line, over the Genesee River Gorge at Portageville, New York, just south of Rochester. The location is one of the most breathtaking in New York state, in the middle of Letchworth State Park.

The Erie Railroad, under the direction of Superintendent Silas Seymor, built the first bridge to cross the river at this location. Commencing construction in 1850, the first train crossed on August 9, 1852. The bridge was built of timber, based on a design suggested by a young boy which doubled up timbers during construction to allow for the removal of any timber for repair in the future. The concept was sound and the bridge became the largest wooden structure in the world, with more than 300 acres of pine timber, totaling more than 1,600,000 board feet, used in its construction. The original construction cost was $175,000. Unfortunately the bridge had an Achilles' Heel, . . . FIRE!

The railroad was acutely aware of the bridge's potential frailty and contracted for the design and construction of a new wrought iron truss bridge. The year was 1875, the component parts for the new bridge were in route when the inevitable happened . . . a fire started that engulfed the bridge in a conflagration that lit the night as bright as day.

The "new" bridge was assembled on the footings of the timber trestle in only forty-seven days! Numerous repair and steel strengthening projects have been made over the years, yet the basic structure remains and is still in active rail service.

*Sr. Structural Inspector, Conrail

Figure 2. The first step. Conrail bridge inspector Carmi Guyette starting to rappel from the top of Portage Bridge. From deck level to the base of the falls is more than 300 feet.

In 1992 Conrail initiated an innovative training program utilizing state of the art, rock climbing techniques and equipment for bridge inspection. The program was developed by Senior Structural Inspector Steve Ross, of Conrail's Structures Department. He pursued its implementation based on his personal knowledge and experience as an avid mountain climbing enthusiast, knowing full well that a properly trained and equipped inspector could better access bridge components in complete safety.

The initial training of inspection personnel consisted of one half day of classroom and two and one-half days of field training. A basic introductory type of course presentation including knot tying and general applications was taught. Practical application on different types of bridges gave inspectors the basics needed to safely implement the program. After successfully completing the initial orientation program, the inspectors were each equipped with approximately one thousand dollars worth of equipment.

In the spring of 1994, a hands on inspection of the entire Portage bridge was scheduled, one hundred and eighteen years after the "new" bridge was built. The bridge inspection team from the Albany Division, together with Conrail's system bridge inspection staff, set out to inspect every inch of the bridge looking for potential defects. The results of the inspection would become the basis for shaping long range future repair and maintenance plans for the bridge.

The inspection was staged from track level with inspectors suspended from dual 11mm ropes! The entire inspection was conducted without interruption to train traffic, which had the benefit of allowing the observation of individual components up close while under load.

Two basic processes were used to move around the bridge utilizing the climbing equipment, belaying and rappelling. "Belaying" is the process of moving horizontally across the structure while on a tether line protected by a climbing partner, while "rappelling" is the process of moving from the highest point to the lowest by sliding down a pair of ropes using friction devices to control the rate of descent.

The west tower of the bridge is the shortest at 47 feet tall. The entire team rappelled down this tower to ensure that everyone was confident with the procedures to be used. Despite their training and experience, a heightened level of apprehension was felt by all when inspecting the 236 foot towers which are adjacent to the 70 foot waterfall. The effect was one of leaning backwards off a 30 story building. The Supervisor of Bridge Inspection, Bob Schmid, was understated when he said, "To back yourself off into oblivion at 236 feet is a little scary."

Four climbers worked their way down each tower leg simultaneously, one man per tower leg, allowing them to keep in eye contact with each other. The mobility of the inspectors allowed them to slide down at a controlled rate, then stop at any location desired and "lock off ".Once the inspector was in a locked position, he had both hands free to examine the individual bridge members, take notes or photographs. He could easily move horizontally by swinging sideways, accessing the diagonal tie rods which would have been very difficult by any other means.

While climbing, the inspectors had a unique vantage point for inspecting the bridge and to observe the beauty of the thousand acre park in which it's located. The inspectors also drew an awestruck crowd of hikers and picnickers, who had the unexpected pleasure of watching the climbers "fly through the air with the greatest of ease".

One unique issue which presented itself during the inspection was impaired communications. Due to the roar of the waterfalls and great height, talking with the climbers would be impossible. Even hand signals would have been impractical. The solution was a high tech, hands free "bone phone". Using a small ear plug which acted as both microphone and receiver, the team was able to talk without the interference of the background noise from the water.

When the inspection was completed, there was a sense of camaraderie and accomplishment felt by the team. The team members included Conrail's Albany Division Bridge Inspectors Dave Cook, Neil Daniels, Bill Garcia, Carm Guyette, Joe Hollander, Rick Packer, Barry Pekarsky, Dave Scatko, Jim Thomas, Supervisor of Inspection Bob Schmid, System Steel Inspector Willy Gall, and System Structural Inspectors Steve Ross and Paul Roddy.

Figure 3. Inspectors Joe Hollande and Paul Roddy rappelling down the tower legs.

Figure 4. Inspector Carmi Guyette setting up his rapel rack. This is the friction device he will be using to control his descent down the dual 11mm ropes.

Figure 5. Inspector Dave Scatko sets out protection for the rappel ropes to ensure their not being damaged by abrasion.

This inspection proved to be safe, thorough and cost effective. With training and the right equipment, Conrail's inspection team has gained the kind of access to high steel that only "Spider Man" might have been able to obtain. The mountaineer style of bridge inspection has been a very successful advancement in Conrail's bridge management program, meeting the rigorous needs of unrestricted mobility, complete safety and the assurance of uninterrupted train traffic.

Proposed 1997 AREA Manual and Portfolio Revisions

The following proposed Revisions of the AREA *Manual for Railway Engineering and Portfolio of Trackwork Plans* have been recommended to the association by the technical committee responsible for each after a letter ballot is approved by: (1) a two-thirds majority of the eligible members voting, and (2) by at least fifty percent of the total eligible voting members on the committee. They are being published here for comment by the general AREA membership and any other interested parties. Comments should be sent to AREA headquarters by December 1, 1996. These comments will be considered by the AREA Board of Direction in deciding whether to give final approval for inclusion of the proposed changes in the Manual and Portfolio Revisions, which if approved, go into effect August 1, 1997.

Proposed 1997 Manual Revisions to Chapter 1—Roadway and Ballast

Part 2—Ballast

Page 1-2-11, Article 2.4.1.9. Change to read:

2.4.1.9 Percent of Flat or Elongated Particles

The percent of flat or elongated particles shall be determined in accordance with the ASTM Standard Test Method, designated D4791. The dimension ratio used in this test shall be 1:3.

Page 1-2-12, Table 2-1. Recommended Limiting Values of Testing for Ballast Material. Change line 7 and Note 1 to read:

Flat and/or Elongated Particles	5.0%	5.0%	5.0%	5.0%	5.0%	5.0%	5.0%	D4791

Note 1—Materials having gradations containing particles retained on the 1" sieve shall be tested by ASTM C535. Materials having gradations with 100% passing the 1" sieve shall be tested by ASTM C131. Use grading most representative of ballast material gradation.

Page 1-2-17, Article 2.10.3. Commentary. Insert the following new paragraph m. after paragraph l. and re-letter the remaining paragraphs:

m. The test for flat or elongated particles is determined by ASTM test method D 4791 using one of three dimension ratios. Track stability can be enhanced by eliminating flat or elongated material in excess of the specification by defining a flat or elongated particle as one that has a width to thickness or length to width ratio greater than three.

Part 4—Culverts

Part 1-4-35, Table 4-15. Values of Coefficient of Roughness (n) for Standard Corrugated Metal Pipe (Manning's Formula). Replace the present table with the following:

Table 4-15
Values of Coefficient of Roughness (n) for Standard Corrugated Metal Pipe (Manning's Formula)

Corrugations	Annular 2⅔ x ½ in.	Helical 1⅓ x ¼ in.			Helical 2⅔ x ½ in.							
Flowing:	Diameters	8 in.	10 in.	12 in.	15 in.	18 in.	24 in.	30 in.	36 in.	42 in.	48 in.	54 in. and larger
Full Unpaved	0.024	0.012	0.014	0.011	0.012	0.013	0.015	0.017	0.018	0.019	0.020	0.021
Full 25% Paved	0.021						0.014	0.016	0.017	0.018	0.020	0.019
Part Full Unpaved	0.027			0.012	0.013	0.015	0.017	0.019	0.020	0.021	0.022	0.023
Flowing:	Pipe Arch				17 x 13	21 x 15	28 x 20	35 x 24	42 x 29	49 x 33	57 x 38	64 x 43 and larger
Full Unpaved	0.026				0.013	0.014	0.016	0.018	0.019	0.020	0.021	0.022
Part Full Unpaved	0.029				0.018	0.019	0.021	0.023	0.024	0.025	0.025	0.026

Flowing:	Annular 3 x 1 in.	Helical 3 x 1 in.							
		36 in.	42 in.	48 in.	54 in.	60 in.	66 in.	72 in.	78 in. and larger
Full Unpaved	0.027	0.022	0.022	0.023	0.023	0.024	0.025	0.026	0.027
Full 25% Paved	0.023	0.019	0.019	0.020	0.020	0.021	0.022	0.022	0.023

Flowing:	Annular 5 x 1 in.	Helical 5 x 1 in.					
		48 in.	54 in.	60 in.	66 in.	72 in.	78 in. and larger
Full Unpaved	0.025	0.022	0.022	0.023	0.024	0.024	0.025
Full 25% Paved	0.022	0.019	0.019	0.020	0.021	0.021	0.022
All pipe with smooth interior*	All Diameters 0.012						

*includes full paved pipe, concrete lined pipe and spiral ribbed pipe.

Part 5—Pipelines

Page 1-5-26, Article 5.4. Specifications for Overhead Pipelines Crossings. Add the following new article:

5.4 Specifications for Overhead Pipelines Crossings

5.4.1 Scope

This section shall govern the design of pipelines which cross the tracks or right-of-way of a Railway Company on overhead structures. It shall include pipelines attached to existing or new vehicle or pedestrian bridges, and existing or new bridges designed for the exclusive use of pipeline facilities. It shall apply to pipelines designed for all levels of operating pressures, to include vacuums, and to all commodities, flammable and non-flammable, usually transported through pipelines.

5.4.2 General Conditions

5.4.2.1 Location Investigation

Where possible, pipelines shall be installed underground. There may be circumstances that warrant consideration of an overhead pipeline crossing; however, an overhead crossing of the tracks or right-of-way of a Railway by a pipeline facility will be investigated for permitting only in the case where the applicant can demonstrate it has exercised due diligence in locating a subgrade crossing.

5.4.2.2 Use of Existing Structures

In no case shall a bridge or other overhead structure of a Railway be used for the attachment of a pipeline facility. Applications proposing to make an attachment to an overhead structures of another shall submit evidence that the owner of the structure has reviewed the plan and has issued, or proposes to issue, a permit or license for the facility.

5.4.3 General Design Requirements

5.4.3.1 Leak Protection

The design shall provide for protection of the property and track structure of a Railway in the event of a pipeline leak or failure by use of a casing pipe or other means acceptable to the Railway.

The design shall direct leaking liquid and dense gaseous products off the Railway right of way, but in no case, less than 25 feet beyond the back of parallel roadway ditches.

5.4.3.2 Emergency Shut-Off Valves

Accessible emergency shut-off valves shall be installed within an effective distance on each side of the Railway right of way. The Engineer may, at his option, accept existing automatic control stations as suitable emergency shut-off valves.

5.4.3.3 Other

Emergency telephone numbers shall be clearly posted on both ends of an overhead pipeline crossing, and pipelines and pipeline bridge structure must have effective apparatus to prevent unauthorized access.

Additional pipeline attachments to an existing overhead pipeline crossing shall be approved by the Railway.

5.4.4 Structural Elements

The structural elements of an overhead pipeline facility shall be the casing pipe, the carrier pipes, attaching hangers or bearings and the supporting bridge.

5.4.4.1 Pipeline Design

5.4.4.1.1 Casing Pipes

Casing pipes shall be assumed to provide no structural support to the carrier pipe. The dead load of the casing pipe shall be included in the calculation of the dead load of the carrier pipe, and the load effects of wind and ice on the carrier pipe shall be calculated with respect to the diameter of the casing pipe.

5.4.4.1.2 Carrier Pipes

Carrier pipes shall be designed in accordance with the most restrictive applicable federal or local regulation for the operating pressure and commodity of the facility. In addition to the loads exerted on the pipe by the conditions of its operation, the structural loads resulting from suspension or bearing conditions shall be considered.

5.4.4.2 Supporting Bridge Design

5.4.4.2.1 General Design Considerations

Bridge spans, bents or piers and foundations shall be designed in accordance with generally accepted engineering practice, accounting for all dead, live, impact, seismic and secondary force loads. Pipe hanger and bearing attachment device design shall consider thermal expansion and seismic displacements.

Drawings, plans, calculations and other documents representing the details of an application shall be prepared, signed and sealed by a professional engineer registered to practice in the state of the installation.

5.4.4.2.2 New Overhead Pipelines Bridges

Overhead pipeline bridges shall be designed in accordance with the following criteria:

Clearances shall be:

VERTICAL—Not less than 25 feet above highest top of rail of the tracks to be spanned, except that cable supported spans shall have a vertical clearances of not less than 50 feet.

HORIZONTAL—Not less than 25 feet from the centerline of the nearest existing main, siding, spur, or industry track, except in cases where the Engineer directs clearance for future tracks is to be allowed.

New beam span, girder and truss type structures and the details of the proposed attachment shall be designed in general accordance with Chapter 15 of this *Manual of Recommended Practice.*

New cable suspended type structures shall be reviewed only upon special application to the Railway. Such application shall identify the design specifications to be used, to include the loads, allowable stresses and design considerations to be applied. Unless other directed by the Engineer, cable supported spans shall include a minimum floor system with lateral bracing.

5.4.4.2.3 Attachments to Existing Overhead Bridges

Where the pipeline is to be attached to an existing overhead structure not specifically designed for pipelines, the following shall apply:

Existing structures proposed as support for a pipeline shall be investigated for the additional loads of the operating pipeline facility. Additionally, the report of the professional engineer shall contain a conclusion with respect the effects of the additional loads on the existing structure.

Pipelines shall be installed inside the main structural members of the supporting bridge. In cases where this is not practical, the pipeline may be attached to the outside surface of the structure, but in no case shall the bottom of the pipeline be less than one foot above the elevation of the lowest main structural member of the supporting bridge.

Pipe hanger and bearing attachment devise design, and their connections to the supporting structure and the pipeline, shall be based on unit stresses equal to one-half (1/2) those otherwise permitted. Attachment device design shall consider thermal expansion, live loads deflection of the existing bridge, and seismic displacements.

Pipeline and attachment designs shall consider the force and effect of the elements of the weather. Attachments shall be protected against corrosion in situations where chemical ice removal is utilized, or other corrosive condition is known or suspected.

5.4.5 Inspection and Maintenance

Overhead pipelines, attachment devices, and supporting structures should be inspected and maintained on a routine basis. And, emergency response procedures should be developed to handle a situation in which an accident or incident might jeopardize the integrity of pipeline facility.

Bibliography

Pages 1-B-1 and 1-B-2. Update the following Bibliography references:

1. American Iron and Steel Institute, "Handbook of Steel Drainage and Highway Construction Products," Fifth Edition, 1994.
2. Federal Highway Administration, "Hydraulic Design of Highway Culverts," Hydraulic Design Series No. 5, September 1985 (As Revised by May 1992 FHWA Memorandum).

Proposed 1997 Manual Revisions to Chapter 4—Rail

Part 2—Specifications

Page 4-2-10, Table 2-4. Section Tolerance. Delete "(Thousandths)" from column heading. Add the following line of information:

Description	Plus	Minus
Head Crown Radius (115 RE, 132 RE, 133 RE, 136 RE and 140 RE.)	2	2

Page 4-2-18, Article 2.1.11 Length. Revise present paragraphs b. and c. as follows:

b. Up to 10 percent of standard length rail of the total tonnage accepted from each individual rolling will be accepted in shorter lengths as follows: 79'-78'-77'-75'-70'-65'-60'-39'-38'-37'-36'-33'-30'.

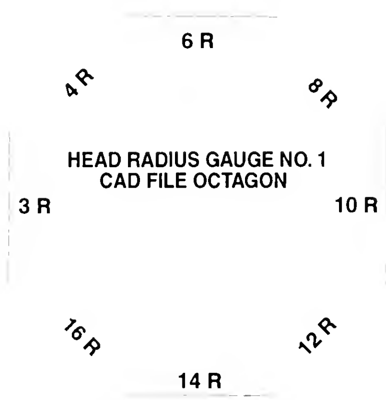
c. Variations from the specified length will be permitted as follows:

	80 ft.	39 ft.
Undrilled	±3 in.	±2 in.
Drilled one end	±3 in.	±2 in.
Drilled both ends	±$\frac{7}{8}$ in.	±$\frac{7}{16}$ in.

Rationale: The proposed changes recognize the fact that most rail is used in welded track where length tolerance is not as critical as in bolted track.

Page 4-2-51. Add Head Radius Gage, Figure 2-47, as follows:

All unspecified radii = 3/16 Area = 18.492



OCTAGON 2" ON EACH SIDE X 45"

Tolerance: Plus or minus (+/–) 2 inches for crown radius of 10 inches.

Method:
1. Place the plus 12R gage on the head of the rail. Contact should be realized in center of gage.
2. Place the minus 8R gage on the head of the rail. Contact should be realized on each side of the gage with daylight visible at the center.

Rationale: Method is simple to use. No need to fumble around with a nominal radius gage and a 0.005" feeler gage. The tolerances are very close to the desired +/– 0.005" previously discussed and demonstrated on the CADCAM blow-up.

Proposed 1997 Manual Revisions to Chapter 16—Economics of Railway Engineering and Operations

Part 1—Railway Location

Page 16-1-1, Part 1. Replace the current Part 1 with the new Part reproduced hereafter:

Part 1—Railway Location

CONTENTS

1.1 Basic Considerations

1.1.1 Relevance of Location

Although railway mileage has been shrinking, at least in the United States and Canada, opportunities for railway location still arise. There are several reasons which may make it necessary or desirable to relocate a segment of line or to build a new line or an extension.

(a) Relocation of a segment of an existing line may be required for external reasons, i.e., because an outside party will acquire the land on which the segment is located, either through negotiation or by condemnation. In either case it is important that the railroad make a complete inventory of the property to be taken and its condition as well as detailed records of the cost of the new line and any temporary or permanent change in operating cost caused by the relocation before any work is undertaken. Salvageable material may be included in the acquisition or may be retained by the railway for use elsewhere. The railway may take the opportunity to include betterments, such as heavier rail, on the relocated line.

(b) New lines are built to reach new sources of traffic such as mines, power plants, manufacturing facilities, industrial parks or waste landfills.

(c) Relocation or realignment may be made for internal reasons. A relocated line may provide better profile or alignment making possible worthwhile increases in trainload or speed, or making room for additional main track, reducing congestion. A relocation or extension may enable a railway to gain better access and provide better service to existing sources of traffic.

(d) In Europe and Japan new lines have been and are being built as very high speed passenger lines. Similar projects are in various stages of consideration or planning in the United States.

When a relocation is made for external reasons, the other party can be expected or required to make the railway whole, to pay for a new line segment as good as the old or, if this is not feasible, to compensate the railway for the future increase in operating expenses (or, more rarely, loss of revenue). When the reason for a contemplated relocation, extension, or new line is to gain new business or to reduce operating expenses or improve service, the required capital expenditure must be evaluated in terms of the decrease in expenses or increase in revenue as discussed below in Sections 1.1.3 and 1.1.4. In the case of high speed passenger lines or relocations for commuter service, probably to be built and frequently to be operated with public support, the revenue includes not only the passenger fares but also the estimated value of public benefits, such as reduced highway and airport congestion.

1.1.2 Definition of Location

A railway line is said to be located when its position is defined in relation to the generally accepted geodetic and geographic references of the area and at some stage staked out on the ground.

The refinement of this definition depends upon the precision requirement of the designated level of planning, i.e., projected, reconnaissance, preliminary, final, or constructed.

The line geometry is defined by the specified limits of its parameters, i.e.—

—ruling grade: the grade which limits the maximum weight of train which can be hauled by a given locomotive and which may not be the same as the maximum gradient if the latter is so short that an entire train is not on it at the same time or if its effect is reduced by momentum;

—compensation for curvature: the reduction in grade on curves to offset curve resistance.

—maximum rate of change in vertical direction;

—minimum tangent distance between points of vertical curve and between points of spiral curve;

—spiral formulas;

—and minimum clearances.

Easy grades and curves and ample tangents and spirals make possible heavy trains, high speeds, and smooth train handling. For relocations or extensions of short or moderate length train size or running times may be controlled by the characteristics of the rest of the line so that expenditures to permit heavier trains or higher speeds on the new segment are not justified. The achievement of easy grades and curves and ample tangents and spirals may be impeded by limitations on the maximum cut and fill having economic justification and the cost of bridges and tunnels. Other limiting factors are geotechnical varying from solid rock to unstable soils, liability to flooding, the preemption of land by public facilities such as highways, and the high cost of land. There are also environmental constraints such as avoidance or protection of wetlands and protecting residential areas from noise.

Geographic factors are topography, streams, geotechnical features (stability and load-carrying capacity of earth and rock), current and prospective land use, environmental considerations such as damage to animal habits or noise in residential areas, and existing structures.

Good railway line location is an economic choice of alternatives, weighing the capital cost of various possible lines against the revenues and expenses to be expected when these various possible lines are operated. The word, "revenue," may be broadly interpreted. In the case of a facility built for a public purpose, such as a commuter line or a high speed passenger line, the benefits of not having to build highways, or reduced congestion and pollution, of time savings to the public, and of increased real property values are properly included. Even in the case of a private enterprise railway company it is possible that improved transportation may enhance the value of some real estate owned by the company.

The capital cost includes the costs of land, grading, structures, track, signal and communication facilities, avoidance of injury to the environment, and regulatory costs.

Revenues include the revenue from traffic gained or retained by new or improved access to customers and by improved services. It is also possible that the improved transportation may enhance the value of some real estate owned by the railway. Expenses are generally the cost of train operation, including equipment maintenance and the costs of maintaining the line.

In the weighing of capital cost against anticipated changes in revenues and expenses, allowance must be made for risk. Projects are not always completed on time and within budget. Geotechnical conditions may not be as favorable as expected; regulatory impediments may arise. Even more subject to risk are projections of revenues and expenses; business may not be as great as expected; competitors' response to improved service may be more effective than anticipated. If volume of traffic is less than expected, operating expense savings will also be less. It may be that other factors can also limit operating savings. For example, reduction in the number of daily trains made possible by a lower ruling grade may not be achieved because service obligations dictate the running of frequent trains.

1.1.3 Principles of Economic Design

The principles of economic design have been summed up in four rules, stated by A. M. Wellington in his book titled "The Economic Theory of the Location of Railways."

(a) "Regardless of how profitable a line promises to be, no expenditure over the absolute minimum is justifiable which is not of itself a profitable investment."

(b) "Conversely, an increase of expenditure which is profitable of itself should always be made."

(c) "The exception to the rule immediately above is that no additional expenditure is wise which endangers completion of a project with the funds available."

(d) "Unless the traffic volume can be predicted quite exactly, it is often best to postpone any expenditure where that can be done without great loss."

These principles, published a century ago, are still valid today and apply to relocations and extensions as much as to new lines. These principles imply that a project should be, to the extent feasible, analyzed in terms of the absolute minimum required and of increments to that minimum. An increment should be included in the project only to the extent that it is itself profitable, i.e., that it confers an increase in benefits sufficient to justify its increase in capital cost. Since capital funds are limited, the most profitable increments should be included first, and no increment should be included which, by its capital needs, imperils the completion of the project with the funds available. The fourth principle, (d), may seem futile; when can we ever, "predict traffic volume quite exactly?" The principle simply means: build small and expand later when, and if, the anticipated traffic materializes.

As an example of the fourth principle, a double track bridge is somewhat cheaper than two single track bridges, but if one single track bridge will suffice for several years until the traffic grows, the interest saved by postponing the expenditure will offset the greater cost of two single track bridges, and if the traffic does not grow the one single track bridge will continue to be enough. On the other hand, if reasonably likely growth in traffic justifies building a line with a low ruling grade, it would be wasteful to build a cheaper line with a higher ruling grade only to abandon it in a few years.

Obviously, a quantitative evaluation of probabilities, a risk analysis, is required, and not just an intuitive judgment. One point to be considered in the risk analysis is the possibility that future expansion to provide capacity for possible growth in traffic may be costly or even impossible because, perhaps, land available now may, in the future, increase greatly in price or be taken for other uses.

Location can contribute to increasing or retaining revenue through improved access to sources of traffic and through facilitating more competitive, i.e., more reliable and speedier service.

The costs of train operation increase, first of all, with the number of trains required to carry the traffic and hence chiefly with ruling grade. Adding more locomotive units to a train can, to a considerable extent, compensate for heavy grades but it is an expensive cure. The extent of savings from reducing the ruling grade depends on the volume of traffic. Increasing the tonnage rating from, say, 6,000 tons to 9,000 tons will save one train a day if the daily traffic is 18,000 tons, two trains a day if it is 36,000 tons, and so on.

Operating costs are also affected by grades other than the ruling grade and by curvature. Curvature is especially burdensome where speed is required as is particularly the case with intermodal and passenger operations. To maintain competitive running times in spite of curvature requires, if it is possible at all, rapid deceleration and acceleration and perhaps higher speeds on more favorable sections of the route. The costs of maintaining the line increases with bridges required, length of line, and geotechnical conditions as well as with curvature and grades and the speed and number of trains operated, i.e., with the volume of traffic.

Because both capital requirements, especially for capacity, and revenues and expenses depend on volume of traffic, it is important that every effort be made to arrive at reliable estimates of future traffic and of the probability, risk, of specified greater or lesser traffic volume. Risk analysis of the other factors, capital costs and operating and maintenance expenses must also be made.

At least qualitative evaluation must be made of geotechnical risk, possible alternative design options, future environmental risk and regulatory risk. The future introduction of new technology should also be considered. New types of equipment may make grade reduction or curve reduction less important.

Easy grades and curves and ample tangents and spirals make possible heavy trains, high speeds and smooth train handling. For relocations or extensions of short or moderate length train size or running time may be controlled by the characteristics of the rest of the line so that expenditures to permit heavier trains or higher speeds on the new segment are not justified. The achievement of easy grades and curves and ample tangents and spirals may be impeded by limitations on the maximum cut and fill having economic justification, and the cost of bridges and tunnels. Other limiting factors are geotechnical, varying from solid rock to unstable soils; liability to flooding; the preemption of land by public facilities such as highways; legal restrictions on the acquisition or use of land; and the high cost of land. There are also environmental constraints, enforced by law or political realities or required as good public relations, such as avoidance or protection of wetlands and protecting residential areas from noise. Any other legal, regulatory, and political constrains must be taken into account. The avoidance or elimination of road crossings may be an important or even the principal consideration. Tax effects of investment and operation must be taken into account.

The requirements for high-speed lines, such as the TGV in France, are quite different from those of conventional railroads. To permit high speeds, curves must be much more gentle. On the other hand, because the train must have a high power-to-weight ratio, and because of the great momentum at high speeds, grades, especially not-very-long grades, can be far steeper, and length of line is much more significant, than on conventional railroads.

1.1.4 Investment Evaluation

The following discussion is only a summary of the investment evaluation process. For more detail as well as discussion of the advantages and disadvantages of each method see Section 1.1.7 References. The financial staffs of the larger railways should be experienced in the evaluation of capital projects.

Estimated time of construction including land acquisition and the timing and amount of funds expended prior to the completion date as well as the lag in the amount of cash recoverable from salvage of the abandonment should be considered, and calculated.

(a) Proposed new lines, extensions, or relocations must be judged in the same way as other proposed capital expenditures, such as those for equipment or new technology.

(b) Relocations to improve service in order to attract or retain traffic or to reduce expenses can be compared with other capital expenditures for the same purposes, such as new, more efficient motive power and, of course, weighed against retaining the status quo.

(c) Strategic considerations, as for new lines or extensions into new territory, may be a primary reason for a project but are particularly difficult to quantify. New lines and extensions are evaluated in terms of the net earnings of the traffic they reach, giving weight, in some instances, to the possibility of competitive retaliation which may take away traffic or force rates down; it may be unwise to short-haul one's connections.

(d) Construction cost levels likely to be in effect during the time the project is being carried out and the costs of construction alternatives must be estimated. The cost of capital, interest on debt (which may well differ from the interest rate on equipment debt) and necessary return on equity must be taken into account. Allowance must be made for ad valorem taxes (or whatever tax method is actually used by the state in which the project is located), income taxes, and income tax deferrals.

(e) The availability of suitable computer programs makes financial calculation so quick and inexpensive that a reasonably sophisticated financial analysis can be made of every plausible alternative.

There are a number of measures used in evaluating capital investments. Among them are: internal rate of return, widely used in the railroad industry; project payout period; net present value, to be compared with amount or present value of investment; profitability index; and benefit/cost ratio, widely used in government for public projects.

Internal rate of return is the discount rate which the railroad would have to apply to future cash flows in order to be indifferent between undertaking the project and not undertaking it.

$$\sum_{i=0}^{n} \frac{B_i - C_i - I_i}{(1+r_d)^i} = 0$$

Project payout period is the number of years over which the sum of annual net benefits equals the investment or capital costs of the project.

$$\sum_{i=0}^{n} B_i - C_i = \sum_{i=0}^{m} I_i$$

Net present value is the present value of net benefits, to be compared with investment.

$$\sum_{i=0}^{n} \frac{B_i - C_i}{(1+r)^i}$$

Profitability index is the ratio of net present value to present value of investment.

$$\frac{\sum_{i=0}^{n} \frac{B_i - C_i}{(1+r)^i}}{\sum_{i=0}^{m} \frac{I_i}{(1+r)^i}}$$

Benefit/cost ratio is the ratio of benefits to costs when both are reduced to present values.

$$\frac{\sum_{i=0}^{n} \frac{B_i}{(1+r)^i}}{\sum_{i=0}^{n} \frac{C_i + I_i}{(1+r)^i}}$$

In these expressions:

i is year

r_d is internal rate of return expressed as a decimal

r is discount rate, expressed as a decimal, applied to future cash flows (either the railroad's cost of capital rate for fixed facilities or a higher "hurdle rate" set by management as a minimum required for new projects).

n is the number of years in a project's life

B_i is the total benefits in year i, including tax savings, if any, salvage values, and, in some cases, estimates of the cash equivalent of non-cash benefits such as the enhancement of the value of land.

C_i is the total cost in year i, including increased taxes if any, and, in some cases, estimates of the cash equivalent of non-cash costs such as reduction of the value of land, excluding the capital expenditure for the project.

I_i is the capital expenditure for the project in year i, the investment in year i.

m is the number of years to complete the project.

(f) Investment evaluation is not as cut and dried as these precise formulas imply. All of the numbers, even the investment cost, the cost of construction, as well as the time required for construction, and the time required to realize benefits, including salvage, are estimates of future expenditures or receipts and their timing. Obviously the level of confidence in the result of the computation depends on the level of confidence in the various inputs. There are methods for quantifying the level of confidence in the result but they depend on quantitative measure of the levels of confidence in the various inputs, measures which are themselves uncertain estimates.

1.1.5 Basic Principles of Location

The following brief points summarize the basic principles which should guide location projects.

(a) Management should provide or approve a clear statement of purpose and need for the new line, extension. or relocation, including economic and strategic reasons.

(b) There should be recognition of the overall transportation system involved, of which this location is usually only a part. In particular, a relocated segment should not impose tonnage or clearance limitations which would adversely affect operations on other parts of the system. Conversely, there is no need to expend capital to permit higher tonnages or larger clearances if other line segments already limit them.

(c) The engineer in charge must recognize that his primary responsibility in planning is to develop a proposal which can be economically justified in accordance with criteria established by management (See Sections 1.1.3 and 1.1.4).

(d) Estimate the capacity requirements for a suitable period of years, based on estimates of the volume and classes of future traffic.

(e) New or changing business opportunities and requirements should be taken into account as should other uncertainties in traffic forecasts.

(f) Line extensions to reach traffic sources may involve intermodal traffic. In such cases the presumably higher cost of longer highway movement must be compared with the added rail investment needed to minimize mileage by highway and possibly with the added rail cost imposed by any resulting increase in ruling grade or other factors.

(g) The feasibility of proposed construction must be weighed, and construction's interference with operations must be allowed for and to the extent practicable, minimized.

(h) Establish the parameters of the line geometry best adapted to the terrain and to the economical performance of the service.

(i) Establish standards of design and construction best adapted to the economical performance of the service.

(j) Investigate a sufficient number of alternative solutions.

(k) The final test is the investment evaluation.

1.1.6 Traffic Forecasts

Locating a railway in effect constitutes designing a physical plant which can produce as economically as possible railroad transportation whose quality of service is commensurate with the nature of the traffic at hand.

Almost all decisions regarding the design and location of a railway depend in some way upon the quantity and class of traffic it is to handle. A railway plant which is satisfactory for a given quantity and class of traffic may not be satisfactory for a different quantity or class of traffic. It is therefore of the utmost importance that estimates of present and future traffic be as complete and accurate as possible. Where substantial uncertainties exist, separate economic analyses for different levels and classes of traffic may be necessary to assess the benefits or losses which would result from major changes in traffic conditions.

A marketing analyst is a necessary member of a line location planning group.

1.1.7 References

1. Wellington, A. M., The Economic Theory of the Location of Railways, John Wiley & Sons, New York, 1887, 1898.
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11. Ernst & Ernst, Rail System Investment Analysis: Financial Analysis of Investment Projects from the Individual Corporate Perspective, Report No. DOT-P-10-78-34, May 1978, National Technical Information Service, Springfield, VA.

1.2 Design Considerations

1.2.1 Initial Construction

When a new line is projected in undeveloped territory which can be a source of only small traffic for a number of years but which can be expected to grow with the stimulus of a railroad, it may be advantageous to introduce extra distance, sharper curves, or steeper grades wherever these are so situated as to be capable of reduction at reasonable expense when justified by growth in traffic. Costs and difficulty in acquiring property for future improvements must be considered. This situation was not unusual in the days of railroad building in this country. It may arise in developing countries today but would be rare in the United States or Canada.

The desirability of each interim construction depends upon whether the interest saved by deferring the construction of the final line will offset the cost of the cheaper work ultimately to be abandoned and the higher train operating costs during the interim. A review of existing and a forecast of future tax policy as related to capital improvements is essential to planning interim construction.

1.2.2 Customer Service

(a) Since freight billing and car supply and control of train movements are now very much centralized, there is little occasion to provide for small stations. However, provision should be made for yard facilities at major traffic generating points. For passenger lines, commuter or high-speed intercity, provision should be made for stations at potential traffic generating points.

(b) If nearby areas are suitable for industrial development, the line should be located and land should be obtained (or optioned) to preserve this potential.

1.2.3 Communications Facilities Required

For surveying and construction, communications facilities required will probably include two-way radio and some form of data transmission or telephone service between construction headquarters and the railroad's headquarters. Telephone service to the Public Switched Telephone Network (PSTN) will also be necessary to communicate offline.

Portable radio transceivers carried by survey party members will be most helpful, as well as keep them in contact with construction offices. At least one base radio station should be provided at the construction headquarters to provide contact with survey crews as well as radio-equipped construction and survey vehicles.

During the construction phase, there might well be need for data transmission including facsimile between construction headquarters, field forces, suppliers and railroad headquarters.

1.2.4 Motive Power

For short relocations and extensions, design should be adapted to the type of motive power (e.g., number of units, horsepower, four motor or six motor) used and to be used on the rest of the line. For longer segments the optimum combination of line and locomotive characteristics can be chosen.

High speed passenger trains require much easier curvature than is acceptable on conventional railroads but, because of their high horsepower to weight ratio and the great momentum resulting from high speed, they can accept much steeper grades.

1.2.5 Terminal/System Operations

(a) For new lines, adequate terminal facilities should be planned, but initially only the minimal portions built.

(b) For line relocations the terminal problem is usually well outside the limits of the new construction. It usually consists of shifting the work load from one terminal to another. Labor problems tend to be the paramount consideration.

Shifts in work load may be temporary or permanent. They may pose the risk of added labor cost but, if properly addressed, they may offer opportunity for improving efficiency or service.

1.2.6 Ruling Grades

(a) Definitions: The ruling grade is that segment of adverse grade which limits the tonnage a locomotive can haul over a line of railroad. It is not necessarily the maximum grade. Momentum grades and grades short enough to permit utilization of a locomotive's short-time rating may be steeper.

(b) Ruling grade is usually the most important consideration in freight line location (excepting only gaining access to traffic) because it determines maximum train size and thereby the number of trains, and hence crew and locomotive cost, required to handle a given traffic.

(c) Ruling grades should be designed uniformly over the entire distance between major traffic generators or train makeup and break-up points.

(d) If there is a possibility of substantial growth in traffic and where it is feasible to do so, sections of ruling grade should be so located as to facilitate future reduction if growth warrants.

1.2.7 Helper Districts

Ideally, locomotives should be working at or near capacity over as much of their district as possible. Where a reasonable balance cannot be made between proposed ruling grades and the rest of the line, it may be best to concentrate adverse grades into one relatively short, steep section. This section can then be operated as a helper district with one or more helper locomotive units on each train.

Helper grades should be such as to permit utilization of the full capacity of the helper and train locomotives on a full tonnage train as determined by the ruling grade on the remainder of the district.

Helper service is less desirable for modern, high-speed freight operation because of the delays (and thus costs as well as impairment of service) associated with cutting the additional engines in and out. These feature are especially significant where the length of train requires that helper engines be cut into the middle of a train. Scheduled fast freight trains should have a high enough ratio of horsepower to weight that they will generally operate at higher than "drag" speed. They may use helpers to maintain speed over the helper district if the increased speed more than offsets the time lost cutting the helpers in and out.

The advantages of helper grades must be weighed against the extra costs of helper crews and locomotives, not always operating from convenient terminals, and against the possible reduction in line capacity caused by the down-grade light engine movements.

Also to be considered is the use of remote control or "slave" helper locomotives that are usually cut in about 1/3 from the rear end of a train. The slave unit(s) are radio controlled from the lead locomotive. Such slave units may be kept in operation over an entire line to not only help the train over the steeper grades, but help sustain higher speeds and help improve train handling, especially train braking.

1.2.8 Balanced Profiles

Usually the total tonnage moving over a line in one direction will not be the same as that moving in the opposite direction. If the ruling grade is the same in both directions, the result will be that locomotives (and thus crews) are not working to full capacity in the light tonnage direction.

Where the extent of this traffic imbalance can be predicted with some confidence it will be advantageous to spend more on construction cost toward reducing the ruling grade in the heavier tonnage direction than in the lighter. The ruling grades should be so balanced that the tractive effort required in each direction, and hence the number of trains, is the same. Note that in computing train resistance (and thus required tractive effort) it is necessary to specify a desired train speed.

1.2.9 Momentum Grades and Locomotive Short-Time Ratings

Momentum grades may often be used to effect economy in construction costs without reducing train size or the over-all operating efficiency of a line. Such grades should not be located at points where train stops or reduced speeds (below that required to operate the grade) are likely to be necessary. The number and character of these grades should in any case be such as to minimize the number and severity of train slack run-ins and run-outs.

Momentum grades should not exceed that grade over which a locomotive loaded for the ruling grade could handle its train in two parts if stalled in the sag.

The length of momentum grades should be such that the maximum speed of tonnage freight trains at the bottom of the sag will not exceed the speed limit for such trains; and such that the minimum speed at the top of the grade will not be less than the speed at which the motive power exerts its maximum tractive effort and in any case should not be less than, say, ten miles per hour to leave some margin.

Short grades, even where momentum cannot be utilized, may exceed the ruling grade if they do not cause the locomotive to exceed its short-time rating.

1.2.10 Compensation for Curvature

Ruling grades should be compensated for curvature so that the total train resistance on curves will not exceed that on tangent track.

1.2.11 Grade Through Tunnels

The grade through long tunnels should be kept as low as is economically practicable (except as noted below) in order to minimize the ventilation problem caused by locomotive exhaust gases and in order to increase the over-all reliability of train operation. It is recommended that in general the grade through tunnels exceeding 1,000 feet in length should not exceed 75 percent of the ruling grade on the district. This reduced grade should preferably be carried some distance above and below the tunnel as well.

A minimum grade sufficient to ensure proper drainage in tunnels should also be established. It is recommended that this be no less than 0.3 percent. A somewhat steeper grade may be in order in wet tunnels where extremely cold weather can cause freezing of the drains.

Summits and sags should be avoided in tunnels without provision for forced air ventilation. The seriousness of the ventilation problem will, in any case, depend upon the type of locomotives used, the number of locomotives in each train, the frequency of train movements and the natural air currents in the vicinity.

1.2.12 Passing Sidings

1.2.12.1 General

Passing sidings should be located on flat grades or on minor summits in preference to sags or the sides of hills. If passing sidings must be located on ruling grades, the grade should be so compensated throughout the entire length of the siding as to permit tonnage trains to be started from a full stop. Where trains must be stopped to operate hand-throw switches at the entrance to and exit from the siding, such compensation should be carried a full train length beyond each end of the siding. Consideration should further be given to the compensation required on the turnout curves at each end of the siding.

1.2.12.2 Siding Length

In determining the total length of passing sidings for freight trains, consideration should be given to the following: maximum length of train, entering speed, margin of stopping distance in siding, length of turnouts to clearance points, and required signal clearance. Highway crossings in sidings should be avoided.

In general, the spacing between passing sidings (and thus the number of passing sidings on a district) should be determined on the basis of the traffic volume (number of trains per unit of time) and the resulting line capacity required, the average delay per train due to meets and passes, and the capital and maintenance costs of the sidings and their associated signaling.

On single-track lines, in the interest of capacity, the time-spacing between adjacent passing sidings should be approximately equal over an entire district between traffic generation, traffic classification, or train make-up points. Time-spacing in this context is the time an average freight train in one direction requires to run between two sidings plus the time an average freight train in the other direction requires to run between the same two sidings.

1.2.12.4 Business or Set-Out Tracks

Where possible, turnouts to auxiliary tracks, such as customer's side tracks, which require switching by road crews should be located between switches on passing sidings.

1.2.13 Signal Design Concerns

While signal system design is not, strictly speaking, a part of location, the locating engineer should, nevertheless, bear it in mind. As noted above, ruling grades should be adjusted to allow for stopping to open and close switches entering and leaving sidings unless the planned signal (traffic control) system makes such stops unnecessary.

The following should be considered as regards train movement control:

- Automatic block signaling with movement orders by voice or digital radio.
- Centralized traffic control.
- Cab signals with or without speed control.
- No wayside signals, but movement orders using voice or digital radio. With digital radio, the locomotive crew can receive movement orders in printed form on printers or computer monitors.

If, as is common, equipment defect detectors are to be installed, thought should be given to locating set-out tracks.

1.2.14 Environmental and Public Policy Concerns

The locating engineer must consider the effects of the environment on the railroad and the effects of the railroad on the environment.

Effort should be made to avoid unstable soils and areas subject to landslides or floods. There may be instances in which land should be avoided because of chemical or radioactive contamination.

The locating engineer should be aware of environmental restrictions such as the statutory protection of wetlands. He should try to forestall complaints of "noise pollution" by keeping some distance away from settled residential areas although this is often impossible. He should give consideration, even at the cost of extra earthwork (preferably fill rather than cut, to avoid drainage problems) and to eliminating highway crossings at grade.

Major considerations are the price of land, and governmental land use planning and restrictions including zoning.

1.2.15 Communications Facilities

The length of a new line or a relocation will affect the communications facilities required. If such a length is 30 miles or less, for example, existing radio base stations might well provide the communications coverage required; or addition of only one or two more radio base stations.

Longer new lines or relocations several miles from an existing line might well require new radio base stations.

On any such project, one should consider using existing communications facilities if sufficient coverage can be obtained. Existing base station sites such as housings, power supplies, antenna towers might be used with only adding radio transmitter-receivers and antennas.

1.2.15.1 Surveys

In making preliminary and final route surveys, communications can be helpful in surveying, mapping and coordination of all personnel involved. Basic two-way voice radio will be required. To provide coverage over a wide area, it might be helpful to use a portable base station to support vehicle and portable units carried by people in the working area. Digital radio might be used in the surveying process to record data and have it sent to a central office for recording and making maps and line plots, etc.

Other surveying techniques use radar profiling, and global positioning satellites, as well as aerial photography.

1.2.15.2 Construction

Two-way radio for voice and data transmission will be required to provide communications between construction crews and project headquarters. A base station will probably be required at the headquarters, and probably more than one frequency will be required to handle the communications load. For example, one frequency might be used between headquarters and the field, and another frequency used by field personnel to communicate with each other.

If contractors are used on the project, they might have their own radio system so that there might be two such systems, one used by the railroad and one by the contractor. Coordination of the two would be essential to avoid any interference.

1.2.15.3 Operations and Maintenance

For operations and maintenance, permanent base radio stations will be required. Also, connections will have to be made to tie the new line's communications facilities into the railroad's existing communications facilities. For example, if the railroad is using dispatcher-controlled radio base stations on the mainline it may well wish to provide such service on the new line.

If the new line serves customers, additional communications facilities may be required to handle voice or digital communications between the customer and the railroad's customer service center.

Also on the operational side, if the railroad has a work-order system in service, it probably would be expanded to include the new line, hence digital radio base stations would have to be installed to provide this coverage along the new line.

If the new line is a major traffic generator or a relocation of a major line, volume communications facilities may be required, such as fiber optic cable or microwave radio, in addition to any two-radio required.

Also, if the new line connects two major mainlines of the railroad or major urban areas, some of the cost of the new line might be offset by leasing right-of-way to a commercial communications company for a fiber optic cable system. In the same vein, electric utility companies might wish to lease right-of-way space for a power transmission line.

1.2.15.4 Other Operating Control Facilities

With automatic equipment identification now in service on many railroads and mandatory tagging of all freight cars operating in interchange, AEI scanners to read freight car and locomotive tags may be desirable. If the new line branches off a mainline, it may be helpful if an AEI scanner were positioned to scan trains entering and leaving the new line. Again, communications would be required to connect the AEI scanners into the existing communications facilities.

Also, the railroad might well desire to install dragging equipment detectors and hotbox detectors on the new line. Here again radio transmitters would be installed at detector sites to provide detector response messages to crews of passing trains. (See Part 5—Economics of Location and Operation of Defect Detector Systems)

If the railroad wishes to operate trains over 50 miles per hour, a block signal system shall be installed, as required by the Federal Railroad Administration. Thus signaling and communications facilities will be required.

1.3 Choice of Design and Alignment

For a short extension such as a spur to serve a nearby industry or a short relocation such as easing a curve, the detailed study discussed below is not practical, although effort should be made to confirm that the net revenue from the industry's traffic or the saving in time and expense and the reduction in hazard resulting from easing the curve justifies the investment.

Before choosing an alignment of a new line or an extension or relocation of any substantial length, the general area where a route is desired should be examined. It is quite likely that two or more possible alignments, differing to a greater or less degree, can be laid out in the area. It is usually wise to examine more than one possible alignment in some detail, following the basic principles outlined in Section 1.1, determining the elements of design described in Section 1.2, and making careful estimates of construction costs. These estimates should be weighed against the estimated revenues and operating and maintenance expenses using traffic projections as described in Section 1.1.6 and investment evaluations as described in Section 1.1.4. In determining the elements of design for a line it will probably be necessary to make repeated changes in detail, weighing added investment in grade reduction, for example, against added net benefits in accordance with the principles stated in Section 1.1.3. The competitive value of time saving in retaining or gaining traffic should not be overlooked.

This examination of alternatives is particularly important when risks, uncertainties, are high. In such case it will be necessary to weigh probabilities. Is the probability of attaining the high estimate of traffic sufficient to justify the added cost of a lower ruling grade?

In planning alignments it is important to give weight to environmental factors and to regulatory, political, and public relations considerations (See Section 1.2.14). The railroad is subject to federal, state, county, and municipal regulation and can be aided or injured by public and government opinion.

In estimating investment, benefits, and costs, the effects of taxation must be given due weight as discussed in Section 1.1.4.

For a large project, design quality will be optimized if specialists in various functions of the railroad, including marketing and operations planning, contribute their expertise.

1.3.1 Traffic and Operating Factors

Estimates of the volume and nature of traffic underlie all the computations of revenue, of the number of trains per day, and of operating and maintenance expenses. In the case of a long-term contract such as for transporting coal to a power plant, the traffic estimate can be quite precise, but in many instances a traffic projection is subject to a wide range of error. In such a case one can work from high and low estimates or simply recognize the uncertainty and not go to too much effort and expense in seeking fictitious precision in estimates of operating and maintenance costs.

Service requirements should be forecast and the location designed to meet them, e.g., by providing for train speed or frequency.

1.3.1.1 Effect of Ruling Grade

From estimates of the volume and nature of traffic and the determination of the ruling grade, the average number of daily trains of each type (heavy unit trains, intermodal, general freight, local freight, etc.) can be calculated, giving weight to any expected seasonality. These calculations in turn make it possible to estimate with some confidence the level of investment justified to reduce the ruling grade.

1.3.1.2 Projected Train Performance (Modeling)

Train performance computer programs take into account train characteristics such as horsepower per ton, general speed restrictions, weight, and length; and line characteristics such as grade, curvature, local speed restrictions, and distance; in order to compute speed at every point on the line; elapsed time, and fuel consumption.

There are also computer dispatching algorithms which can be used to calculate train interference, delay, and congestion; with such a program, siding spacing and location can be evaluated.

1.3.2 Causes of Expenses

(a) From the outputs of these computer programs the locating engineer can ascertain the effects of the various line characteristics on transportation and equipment maintenance expenses. Crew wages are generally proportional to train-miles but specific labor agreements should be checked. Locomotive maintenance is in part proportional to the number of locomotives required and in part to fuel consumption. The Surface Transportation Board (STB) has a computer program, Uniform Railroad Costing System (URCS), which develops such unit costs, although the individual railroad may well have better, more specific data. Car maintenance cost is similarly divided between car-mile cost and car-day cost. Foreign car cost is proportional to car time and mileage, private line car cost to car-miles.

(b) The expense of maintenance of way is in part related to miles of route and in part to gross-ton-miles, a traffic-based factor. It is also related to subgrade conditions, curvature, and grade. The URCS develops estimates of maintenance of way (M of W) and other costs although individual railroads probably have their own much better M of W budgeting programs. Since replacements of newly laid rail and ties will become necessary only years in the future, cash flow projections are desirable.

1.3.3 Effects of Line Characteristics

(a) Ruling grade determines the maximum weight of train and hence the number of trains, and their engines and crews, required to move a projected volume of traffic. While the diesel-electric locomotive with its high tractive force and multiple unit (m.u.) capability has greatly increased train size on grades, the fact remains that ruling grades still limit train weight.

(b) Grades other than ruling grades, rise and fall, do not limit train weight but impose the costs related to fuel consumption. To the extent that braking is not needed on fairly short descending grades, the kinetic energy gained is used in ascending out of the sag, and there is virtually no added fuel consumption. If a descending grade is long enough and steep enough to require braking, energy and fuel are lost (exception: on some electrified railroads, regenerative braking can return energy to the power system). An undulating profile, however, makes train handling more difficult, generates slack action causing shocks to cars and lading and occasional break-in-twos.

(c) Length of line (distance) affects crew-miles, locomotive-miles (and fuel consumption), and car-miles as well as maintenance of way expenses.

(d) Curvature increases rail and wheel wear and, if uncompensated, increases fuel consumption. Rail wear on curves can be ascertained from the railroad's experience. Wheel wear may be estimated to be proportional to rail wear. Line and surface are somewhat more expensive to maintain on curves.

1.3.4 Highway-Rail Grade Crossings

From a liability standpoint, it would be desirable not to have any highway-rail crossings at grade. All highways should be grade separated from the railroad.

It should be noted that the railroad will have to discuss such crossings with the road authority having jurisdiction, which includes the state highway or transportation department.

Proposed 1997 Manual Revisions to Chapter 28—Clearances

Part 3—Methods and Procedures

Page 28-3-30, Table 3-3. Legal Clearance Requirements by State. Replace the present table with the revised tables of both English and Metric units reproduced hereafter:

A.R.E.A. MANUAL FOR RAILWAY ENGINEERING 28-3-25

LEGAL CLEARANCE REQUIREMENTS
(Metric Units)

THIS CHART IS FOR INFORMATION ONLY AND NO LIABILITY CAN BE ASSUMED – CONSULT INDIVIDUAL STATE REGULATIONS FOR MORE DETAILED INFORMATION

			TRACK CENTERS								VERTICAL						HORIZONTAL																						
																							PLATFORMS							SIGNALS: LOW BETWEEN TRACKS		SWITCH BOXES ETC.							
	STATE/REGULATION REFERENCE		MAIN TRACKS	ANY TWO SUBSIDIARY TRACKS	ADJACENT SUBSIDIARY TRACK TO ANY MAIN TRACK	LADDER TRACK ADJACENT TO ANY PARALLEL TRACK	TWO ADJACENT PARALLEL LADDER TRACKS	LEAD REPAIR AND CABOOSE TRACKS	TEAM TRACKS IN PAIRS	UNLOADING TRACKS AT PLATFORMS	GENERAL	THRU BRIDGES	HIGHWAY BRIDGES	TUNNELS	BUILDING DOORS	IN BUILDINGS	GENERAL	THRU BRIDGES	HIGHWAY BRIDGES	TUNNELS	BUILDING DOORS	IN BUILDINGS	A	B	C	D	E	F	HIGH	HEIGHT	CLEARANCE	HEIGHT	CLEARANCE	POLES	MAIL CRANES	ICING DOCKS	ORE AND COAL DOCKS	CATTLE CHUTES	WIRES	PIPELINES
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41
ALABAMA	23CFR 646B												7.01					2.74	2.74																					
ALASKA	NOTE 1																																							
ARIZONA	R14-5-110	06-30-82	4.27	4.27	4.57	6.1	6.1	4.27	3.96	3.96	6.71	6.71	7.01	6.71	5.49[3]	5.49[4]	2.59	2.44	2.59	2.44	2.59[3]	2.59	0.2	1.42			1.22	2.21[10]	2.59	0.91	1.83	0.13	0.91	2.59	E	2.34	2.59	2.59		
ARKANSAS	CASE R-1012	01-01-84	4.27	4.27	4.27	5.18	6.1	4.27	3.96	3.96	6.71	6.71	6.71	7.01	5.18[3]	5.49[4]	2.59	2.44	2.59	2.44	2.13[3]	2.13[5]	0.2	1.52	1.22	1.75	1.22	2.44	2.59	0.91	1.83	0.1	0.91	2.59	2.59	2.03[12]	2.59	2.59		
CALIFORNIA	G.O. 26-D	09-25-77	4.27	4.27	4.57	6.1	6.1	4.27	3.96	3.96	6.86	6.86	6.86	6.88	5.49	5.49[4]	2.59	2.44	2.59	2.44	2.59	2.59	0.2	1.42			1.22	2.29[10]	2.59	0.91	1.83	0.1	0.91	2.59	1.96[11]	2.34	2.59	2.59	6.86 / 2.59	6.86 / 2.59
CANADA	SRRC	05-04-82	3.96	4.11	4.27	4.57	5.49	4.11	3.66	3.66	6.71	6.71	7.01	6.71	6.71	6.71	2.55	2.44	2.55	2.44	2.44	2.44	0.13	1.17			1.22		2.55	1.22	1.75	0.13	1.17	2.55	2.55	2.55	2.55	2.55	6.70	6.70
COLORADO	DEC. 55821	01-10-81	4.27	4.27	4.57	5.18	6.1	4.27	3.96	3.96	6.86	6.71	6.86	7.01	5.18[3]	5.49[4]	2.59	2.44	2.59	2.44	2.13	2.13[5]	0.2	1.52	1.22	1.75[7]	1.22	2.59	2.59	0.91	1.83	0.1	0.91	2.59	2.59	2.03[12]	2.59	2.59		
CONNECTICUT	TITLE 13b	10-03-79	3.96	3.96	4.57	5.49	5.79	4.27	3.96	3.96	6.88	6.86	6.86	6.88	6.86	6.86	2.59	2.44	2.59	2.44	2.44[3]	2.44	0.2	1.57			1.22	2.59[8,10]	2.59	0.91	1.83	0.1	0.91	2.59	1.96[11]	2.59	2.44	2.59		
DELAWARE	NOTE NO 22	03-22-83																																						
DIST./COLUMBIA	TITLE 24	Oct 83	4.27	4.27	4.57	5.49	5.79	4.27	3.96	3.96	6.71	6.71	6.71	6.71	5.49[3]	5.49[3]	2.59	2.44	2.59	2.44	2.44[3]	2.44	0.2	1.42	1.22	1.70[22]			2.59	0.91	1.83	0.1	0.91	3.05			2.59	2.59		
FLORIDA	CHPTR. 14-57	09-27-81	4.11	4.11	4.11	5.49	5.79	4.11	3.96	3.96	6.71	6.71[2]	6.71	6.71	5.18	5.18	2.44[2]	2.44[2]	2.44	2.44[2]	2.44	2.44	0.2	1.42	CFH	1.75			2.44	0.91	1.83	0.1	0.91	3.05	1.96[11]	2.34	2.44	2.44		
GEORGIA	NOTE 1	07-08-88																																						
IDAHO	31.71.01	07-08-83	4.27	4.27	4.57	6.1	6.1	4.27	3.96	3.96	7.18	6.88	6.86	6.86	5.49[3]	5.49	2.59	2.44	2.59	2.44	2.59	2.59	0.2	1.42	1.22	2.71	1.37	2.59	2.44	0.91	1.83	0.1	0.91	2.59	2.59	2.03[12]	2.59	2.44	NOTE 24	
ILLINOIS	TITLE 92 I.A.C.	10-1-87	4.11[16]	4.11[16]	4.57	5.18[16]	5.79[17]		4.11	4.11	6.55	6.46	6.55	6.55			2.44	2.44	2.44	2.44[2]	2.13	2.13	0.1	1.37	0.2	1.55[9]	CFH	1.88	2.59	0.86	2.44			2.74	2.44	2.44	2.44		2.44	2.44
INDIANA	STATUTES	7-4-89									6.71	6.71	6.71	6.71	6.71	6.71	2.44	2.44	2.44	2.44	2.44	2.44							2.44					2.44	2.44	2.44	2.44	2.44		
IOWA	NOTE 1	08-15-87																																						
KANSAS	KSA 66-141	05-01-81	4.27	4.27	4.27	5.18	6.1	4.27	3.96	3.96	6.86	6.71	6.86	7.01	5.18[3]	5.49[4]	2.59	2.44	2.59	2.44	2.13[13]	2.13[13]	0.2	1.52	1.22	1.88[7]	1.22	2.59	2.59	0.91	1.83	0.1	0.91	2.59	2.59	2.03[12]	2.59	2.59		
KENTUCKY	277.240	10-03-82											6.71																											
LOUISIANA	NOTE 1																																							
MAINE	CHPTR. 501	08-15-79									6.71	6.71	6.71	6.71[2]	5.18[3]	5.18	2.44	2.44	2.44	2.44[2]	2.13[3]	2.44	0.3	1.63	1.22	1.75[7]		[9]	2.44	0.97	1.65	0.1	0.91	2.44	1.85[11]	1.83	1.83	2.44		
MARYLAND	COMAR 09.12.91.04	02-04-88	4.27	4.11	4.57	5.49	5.79	4.11	3.96	3.98	6.71	6.71	6.71	6.71	5.49[3]	5.49[4]	2.59	2.44	2.59	2.44	2.44[3]	2.44	0.2	1.55	1.22	1.70[9]	1.22	2.59[8]	2.59	0.91	1.83	0.1	0.91	2.59	1.96[11]	2.34	E	2.59		
MASSACHUSETTS	CHPTR. 160	04-22-71	3.96	3.96	3.96	5.18	5.18	5.18	3.96	3.96	6.88	6.86[2]	6.86	6.86[2]	6.88	6.88	2.59	2.59[2]	2.59	2.59[2]	2.59					1.75[7]			2.59	0.91	1.98			2.59	2.59	2.59	2.59	2.59		
MICHIGAN	ACT NO. 26	11-19-88	4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.27	6.86	E	E	6.86[2]	6.86	6.86	2.59	E	E	2.59[2]	2.59	2.59							E	E	E	E	E	2.59	E	E	E	E		
MINNESOTA	219.46	01-01-80	4.27	4.27	4.27	5.18	5.79	4.27	4.27	4.27	6.71	6.71	6.71	6.71	6.71[3]	6.71	2.59	2.59	2.59	2.59	2.59[3]	2.59							2.59					2.59	2.59	2.59	2.59	2.59		
MISSISSIPPI	NOTE 1																																							
MISSOURI	4CSR 265.8.060	06-30-86	4.27	4.27	4.27	5.18	5.79	4.27	4.27	4.11	6.71	6.71	6.71	6.71	6.71	6.71	2.59	2.44	2.59	2.59	2.59[3]	2.59	0.1	1.37	0.2	1.55[9]	CFH	1.75	2.59	1.22	1.57 TO 2.44	0.08	1.27	2.59	1.96[11]	2.59	2.59	2.59		
MONTANA	ARM. 38.4.401	10-27-76	4.27	4.27	4.57	6.1	6.1	4.27	3.96	3.96	6.86	6.86	6.86	6.86	5.49[3]	5.49[4]	2.59	2.44	2.59	2.44	2.59[3]	2.59	0.2	1.42	1.22	1.75[7]	1.22	2.59[10]	2.44	0.91	1.83	0.1	0.91	2.59	2.59	1.75	2.59	2.59	NOTE 24	
NEBRASKA	REGULATIONS	08-02-88	4.27	4.27	4.57	5.18	6.1	4.27	3.96	3.96	6.86	6.86	7.01	7.01	5.18[3]	5.49[4]	2.59	2.44	2.59	2.44	2.13[3]	2.13[5]	0.2	1.52	1.22	1.75[6]	1.22	2.59[10]	2.59	0.91	1.83	0.1	0.91	2.59	2.59	2.03[12]	2.59	2.59		6.86
NEVADA	G.O. NO 11	05-19-79	4.27	4.27	4.57	6.1	6.1	4.27	3.51	3.96	6.71	6.71	7.01	6.71	5.49[3]	5.49[4]	2.59	2.44	2.59	2.44	2.59[3]	2.59	0.2	1.42			1.22	2.21[10]	2.59	0.91	1.83	0.1	0.91	2.59	E	2.34	2.59	2.59		6.71
NEW HAMPSHIRE	ORDER 6922	12-20-84	4.27	4.27	4.57[C]	5.49	5.49	4.27[18]	3.96	3.96	6.4	6.4	6.4	6.4	5.18[3]	5.18	2.44[14]	2.44[14]	2.44[14]	2.44[14]	2.13[3]	2.44	0.3	1.63	1.22	1.75[7]		[9]	2.59	0.97	1.65	0.1	0.91	2.59	1.85[11]	1.83	1.83	2.59		
NEW JERSEY	NJ DOT	03-18-84										7.01	7.01	7.01																										
NEW MEXICO	ORDER 2202	1957	4.27	4.27	4.57	5.18	6.1	4.27	3.96	0	6.86	6.71	6.86	7.01	5.18[3]	5.49[4]	2.59	2.44	2.59	2.44	2.13[5]	2.13[5]	0.2	1.42	1.22	1.75	1.22	2.44	2.59	0.91	1.83	0.1	0.91	2.59	2.59	2.03[12]	2.59	2.59	6.86 / 2.59	6.86 / 2.59
NEW YORK	NY POLICY	12-19-88	4.11	4.11	4.57	5.49	5.79	4.11	4.11	4.11	6.71	6.71	6.71	6.71	5.49[3]	5.49	2.59	2.44	2.59	2.44	2.44[3]	2.44	0.2	1.55	1.22	1.70[9]	1.22	2.59[8,10]	2.59	0.91	1.83	0.1	0.91	2.59	1.96[11]	2.44	E	2.44		
NORTH CAROLINA	NOTE 1																																							
NORTH DAKOTA	Sect.49-10.1-13	05-18-81									6.4	6.4[2]	6.4	6.4[2]	6.4	6.4	2.44[2]	2.44[2]	2.44	2.44[2]	2.44	2.44					1.22	2.44	2.44					2.44	2.44	2.44	2.44	2.44		
OHIO	ORDER 4963.42-.43	01-20-83	4.27	4.27	4.57	4.57	5.49	4.27			6.4	6.4[2]	6.4	6.4[2]	6.4	6.4	2.44	2.44[2]	2.44	2.44[2]	2.44	2.44							2.44					2.44	2.44	2.44	2.44	2.44		
OKLAHOMA	ORDER 33847	12-03-84	4.27	4.27	4.27	5.18	6.1	4.27	3.96	3.96	6.71	6.71	6.71	7.01	5.18[3]	5.49[4]	2.59	2.44	2.59	2.44	2.13[3,5]	2.13[5]	0.2	1.52	1.22	1.75	1.22	2.44	2.59	0.91	1.83	0.1	0.91	2.59	2.59	2.03[12]	2.59	2.59		
OREGON	ORDER 83-313	06-08-83	4.57	4.27	4.57	6.1	6.1	4.27	4.27	4.27	6.32	6.32	6.32	6.32	5.49[3]	5.49[4]	2.59	2.44	2.59	2.44	2.59	2.59	0.2	1.42			1.22	2.21[10]	2.59	0.91	1.83	0.1	0.91	2.59	2.59	2.34	2.59	2.59	6.83 / 2.59	6.83 / 2.59
PENNSYLVANIA	TITLE 52.33C	08-15-79	4.11	4.11	4.57	5.49	5.79	4.11	4.11		6.71	6.71	6.71	6.71	5.49[3]	5.49	3.66	2.44	3.66	2.44	2.44[3]	2.44	0.2	1.55	1.22	1.70[9]	1.22	2.59[8,10]	3.66	0.91	1.83	0.1	0.91	3.66	1.96[11]	2.44		2.59		
RHODE ISLAND	DOCK 608	11-14-86	3.96	4.27	4.57	5.49	5.79	4.27	3.96	4.27	5.49	5.49	5.49	5.49	5.49[3]	5.49	2.59	2.44	2.59	2.44	2.44[8]	2.44	0.2	1.57			1.22	2.59[8,10]	2.59	0.91	1.83			2.59	1.96[11]	2.59	2.44	2.59		
SOUTH CAROLINA	SEC. 58-15-167.0	04-01-72										6.86						2.44																						
SOUTH DAKOTA	ORDER F-2465	07-03-87									6.86	6.86	6.86[2]	6.86	5.18	6.88	2.59	2.44	2.44[2]	2.59	2.13	2.59	0.2	1.42	CFH	1.75[7]			2.44			0.1	0.91	2.59	2.59	2.59	2.59	2.59	NOTE 24	
TENNESSEE	RULE 1220-3-1-.12	02-08-79	4.27	4.27	4.27	5.49	5.49	4.27	3.96	3.96	6.71	6.71	6.71	6.71	5.18	5.18[4]	2.44	2.44	2.44	2.44	2.44	2.44	0.2	1.42	1.22	1.75	1.22	2.29	2.44		1.96	0.1	0.91	2.44	1.96[11]	2.13	2.44	2.44		
TEXAS	CSA 6558.A-F	07-07-83									6.71	6.71	6.71	6.71[2]	6.71[3]	6.71	2.59	2.29	2.59	2.59[2]	2.59[3]	2.59	0.3	1.37					2.59	0.76	1.68	0.15	1.22	2.59		2.13	2.59	2.59	6.71	6.71
UTAH	G.O. 86	08-31-83	4.27	4.27	4.57	5.18	5.49	4.27	3.51	3.51	6.86	6.86	7.01	6.86	5.49[3]	5.49[4]	2.59	2.44	2.59	2.44	2.59[3]	2.59	0.2	1.42	0.53	1.73[8]	1.22	2.21[10]	2.59	0.91	1.83	0.1	0.91	2.59		2.34	2.59	2.59		6.86
VERMONT	TITLE 30	03-01-81										6.71	6.71				2.44	2.44	2.44	2.44	2.44	2.44	0.2	2.44												E				
VIRGINIA	NOTE 1																																							
WASHINGTON	CHPTR. 480-60	10-08-88	4.27	4.27	4.57	6.1	6.1	4.27	3.96	3.96	6.86	6.86	6.86	6.86	5.49[3]	5.49[4]	2.59	2.44	2.59	2.44	2.59[3]	2.59	0.2	1.42	1.22	2.21[7]	1.37	2.59[8]	2.44	0.91	1.83	0.1	0.91	2.59	2.59	2.21[7]	2.59	2.59		
WEST VIRGINIA	G.O. No 189.3	12-20-83	4.27	4.27	4.57	4.57	5.49	4.27	3.96	3.96	6.71	6.71	6.71	6.71	6.71	5.49	2.44	2.44	2.44	2.44	2.44	2.44	0.2	1.52	1.22	1.93			2.44	0.91	1.83	0.1	0.91	3.05	1.96	2.34	2.44	2.44		
WISCONSIN	SECT 192.31 & 192.53	12-31-83	4.27	4.27	4.27	4.27	4.27	4.27	4.27	4.27	7.01	7.01[2]	7.01	7.01[2]	7.01	7.01	2.59		2.59	2.59[2]	2.59	2.59			0.53	1.83[9]	1.52	1.93	2.59	E	E	E	E	3.66		2.59	2.59	2.59		
WYOMING	CHPTR XIII	08-04-79	4.27	4.27	4.57	5.18	6.1	4.27	3.96	3.96	6.86	6.71	7.16	7.01	5.18[3]	5.49[4]	2.59	2.44	2.59	2.44	2.13[3,5]	2.13[5]	0.2	1.52	1.22	1.75[7]	1.22	2.59	2.59	0.91	1.83	0.1	0.91	2.59	2.59	2.03[12]	2.59	2.59	NOTE 24	5.86

ARCHITECTS, CONTRACTORS ETC. SHOULD CHECK WITH RAILROAD INVOLVED

DIMENSIONS:

ARE SHOWN IN METERS. THEY ARE FOR TANGENT TRACK AND MOST LAWS SPECIFY INCREASES FOR CURVED AND SUPERELEVATED TRACK.

VERTICAL:

MEASURED FROM TOP OF RAIL.

HORIZONTAL:

MEASURED FROM CENTER LINE OF TRACK. APPLIES TO NEW CONSTRUCTION. SOME RECONSTRUCTION AND SOME EXTENSIONS. SOME CAN BE VERIFIED UPON APPROVAL OF APPLICATION BY GOVERNING BODY ARE BASED ON MAXIMUM CAR SIZE FOR ARIZONA, CALIFORNIA, IDAHO, MINNESOTA, MONTANA, NEVADA, NORTH DAKOTA, OREGON, WASHINGTON AND WEST VIRGINIA. ALL ARE MINIMUM EXCEPT COLUMNS 24, 26, 28, 31 AND 33 WHICH ARE MAXIMUM.

HAWAII OMITTED – NO COMMON CARRIER RAILROADS
C = CONDITIONAL – SEE SPECIFIC REGULATION.
H = HEIGHT OF CARS GOVERNS
C F H = CAR FLOOR HEIGHT
E = EXEMPT

COLUMNS:

2 SHOWS BASIC REGULATIONS.
3 SHOWS EFFECTIVE DATE OR DATE OF LATEST AMENDMENT.
7 & 8 APPLY TO HAND AND MECHANICALLY OPERATED SWITCHES, EXCEPT AS NOTED.
12 & 18 PREVAIL FOR ALL ITEMS NOT OTHERWISE PROVIDED FOR.
13 & 19 BRIDGES SUPPORTING TRACKS.
14 & 20 BRIDGES SPANNING TRACKS.
24 & 25 PASSENGER PLATFORMS
26 & 27 FREIGHT PLATFORMS ON SIDE TRACKS AS NOTED.
28 & 29 FREIGHT PLATFORMS ON SIDE TRACKS. STEPPED PLATFORMS ARE NOT GENERALLY ALLOWED
35 OTHER THAN TROLLEY CONTACT POLES.
36 TO CENTER OF STAND EXCEPT AS NOTED
37 APPLIES TO BOTH SUPPORTS AND PLATFORMS, EXCEPT AS NOTED

NOTES:

1. NO CURRENT REGULATION – CHECK WITH RAILROAD INVOLVED.
2. LESSER CLEARANCES NOT PERMITTED IN QUADRANTS.
3. ENGINE HOUSES AND SHOP BUILDINGS EXEMPT, OR PERMITTED LESSER DIMENSIONS.
4. ONLY IF TRACKS END WITHIN BUILDING.
5. MAY BE REDUCED TO 1.75 IF 2.51 (2.59 FOR WYOMING) PROVIDED ON OPPOSITE SIDE.
6. MAY BE REDUCED TO 1.75 IF 2.44 (2.59 FOR NE & WY) PROVIDED ON OPPOSITE SIDE.
7. ONLY IF 2.44 (2.21 FOR MT & WY, 2.59 FOR WA, MA & SD.) PROVIDED ON OPPOSITE SIDE.
8. MAY BE LESS ON ONE SIDE IF FULL GENERAL (COL. 18) PROVIDED ON OPPOSITE SIDE.
9. PASSENGER PLATFORMS ONLY
10. MAY BE 2.44 AT 1.37 FOR REFRIGERATOR CAR PLATFORMS ONLY
11. TO ENDS OF ARMS IN OPERATING POSITION.
12. FOR PLATFORMS AT CAR EAVE HEIGHT – SUPPORTS TO BE 2.44 (2.59 FOR TEXAS)
13. FOR MAIN TRACKS – ALL OTHER 5.71.
14. 2.59 FOR MAIN AND PASSING TRACKS.
15. FOR FREIGHT TRACKS – CAN BE 3.96 FOR PASSENGER TRACKS.
16. FOR HAND OPERATED SWITCHES – MAY BE 4.57 FOR MECHANICALLY OPERATED SWITCHES.
17. FOR HAND OPERATED SWITCHES – MAY BE 5.18 FOR MECHANICALLY OPERATED SWITCHES
18. PARALLEL LEAD TRACKS – 5.49.
19. MAY BE REDUCED TO 1.84 IF 2.51 PROVIDED ON OPPOSITE SIDE.
20. MAY BE REDUCED TO 1.84 IF 2.44 PROVIDED ON OPPOSITE SIDE.
21. MUST HAVE ADDITIONAL SIDE CLEARANCE OF 2.54 CENTIMETERS PER 0.00873 RADIAN CURAVATURE.
22. STATE REGULATIONS CONFORM TO A.R.E.A. STANDARDS. SEE CHAPTERS 14 & 28 FOR SPECIFICS.
23. ONLY IF 2.59 CLEARANCE AND 4.27 TRACK CENTERS PROVIDED ON OPPOSITE SIDE.
24. STATE REGULATIONS CONFORM TO NATIONAL ELECTRICAL SAFETY CODE.
25. ENGLISH UNITS ARE SHOWN IN REGULATIONS. METRIC UNITS ARE A CONVERSION FROM ENGLISH UNITS ROUNDED TO THE NEAREST CENTIMETER

11-15-85

A R E A MANUAL FOR RAILWAY ENGINEERING 28 3 25

LEGAL CLEARANCE REQUIREMENTS
(English Units)

THIS CHART IS FOR INFORMATION ONLY AND NO LIABILITY CAN BE ASSUMED – CONSULT INDIVIDUAL STATE REGULATIONS FOR MORE DETAILED INFORMATION

ARCHITECTS CONTRACTORS ETC SHOULD CHECK WITH RAILROAD INVOLVED

TRACK CENTERS (cols 4–11) · VERTICAL (cols 12–17)

1	2 STATE/REGULATION REFERENCE	3	4 MAIN TRACKS	5 ANY TWO SUBSIDIARY TRACKS	6 ADJACENT SUBSIDIARY TRACK TO ANY MAIN TRACK	7 LADDER TRACK ADJACENT TO ANY PARALLEL TRACK	8 TWO ADJACENT PARALLEL LADDER TRACKS	9 LEAD REPAIR AND CABOOSE TRACKS	10 TEAM TRACKS IN PAIRS	11 UNLOADING TRACKS AT PLATFORMS	12 GENERAL	13 THRU BRIDGES	14 HIGHWAY BRIDGES	15 TUNNELS	16 BUILDING DOORS	17 IN BUILDINGS
ALABAMA	23CFR 668B												23-0			
ALASKA	NOTE 1															
ARIZONA	R14-5-110	[illegible]	14-0	14-0	15-0	20-0	20-0	14-0	13-0	13-0	22-0	22-0	23-0	22-0	18-0	18-0
ARKANSAS	CASE R-1012	[illegible]	14-0	14-0	14-0	17-0	20-0	14-0	13-0	13-0	22-0	22-0	22-0	23-0	17-0	18-0
CALIFORNIA	G O 26-D	[illegible]	14-0	14-0	15-0	20-0	20-0	14-0	13-0	13-0	22-6	22-6	22-6	22-6	18-0	18-0
CANADA	SRRC	[illegible]	13-0	13-6	14-0	15-0	18-0	13-6	12-0	12-0	22-0	22-0	23-0	22-0	22-0	22-0
COLORADO	DEC 85821	[illegible]	14-0	14-0	15-0	17-0	20-0	14-0	13-0	13-0	22-6	22-0	22-6	23-0	17-0	18-0
CONNECTICUT	TITLE 13b	[illegible]	13-0	13-0	15-0	18-0	19-0	14-0	13-0	13-0	22-6	22-6	22-6	22-6	22-6	22-6
DELAWARE	NOTE NO 22	[illegible]														
DIST/COLUMBIA	TITLE 24	Oct 83	14-0	14-0	15-0	18-0	19-0	14-0	13-0	13-0	22-0	22-0	22-0	22-0	18-0	18-0
FLORIDA	CHPTR 14-57	[illegible]	13-6	13-6	13-6	19-0	19-0	13-6	13-0	13-0	22-0	22-0	22-0	22-0	17-0	17-0
GEORGIA	NOTE 1	[illegible]														
IDAHO	31 71 01	[illegible]	14-0	14-0	15-0	20-0	20-0	14-0	13-0	13-0	23-6	22-6	22-6	22-6	18-0	18-0
ILLINOIS	TITLE 92 IAC	[illegible]	13-6	13-6	15-0	17-0	18-0		13-6	13-6	21-8	21-3	21-6	21-6	H	
INDIANA	STATUTES	[illegible]									22-0	22-0	22-0	22-0	22-0	22-0
IOWA	NOTE 1	[illegible]														
KANSAS	KSA 66-141	[illegible]	14-0	14-0	14-0	17-0	20-0	14-0	13-0	13-0	22-6	22-0	22-6	23-0	17-0	18-0
KENTUCKY	277.240	[illegible]											22-0			
LOUISIANA	NOTE 1															
MAINE	CHPTR 501	[illegible]									22-0	22-0	22-0	22-0	17-0	17-0
MARYLAND	COMAR 09 12 91 04	[illegible]	14-0	13-6	15-0	18-0	19-0	13-6	13-0	13-0	22-0	22-0	22-0	22-0	18-0	18-0
MASSACHUSETTS	CHPTR 160	[illegible]	13-0	13-0	13-0	17-0	17-0	17-0	13-0	13-0	22-6	22-6	22-6	22-6	22-6	22-6
MICHIGAN	ACT NO 26	[illegible]	14-0	14-0	14-0	14-0	14-0	14-0	14-0	14-0	22-6	E	E	22-6	22-6	22-6
MINNESOTA	219 46	[illegible]	14-0	14-0	14-0	17-0	19-0	14-0	14-0	14-0	22-0	22-0	22-0	22-0	22-0	22-0
MISSISSIPPI	NOTE 1															
MISSOURI	4CSR 265 8 060	[illegible]	14-0	14-0	14-0	17-0	19-0	14-0	14-0	13-6	22-0	22-0	22-0	22-0	22-0	22-0
MONTANA	ARM 38 4 401	[illegible]	14-0	14-0	15-0	20-0	20-0	14-0	13-0	13-0	22-6	22-6	22-6	22-6	18-0	18-0
NEBRASKA	REGULATIONS	[illegible]	14-0	14-0	15-0	17-0	20-0	14-0	13-0	13-0	22-6	22-6	23-0	23-0	17-0	18-0
NEVADA	G.O NO 11	[illegible]	14-0	14-0	15-0	20-0	20-0	14-0	11-6	13-0	22-0	22-0	23-0	22-0	18-0	18-0
NEW HAMPSHIRE	ORDER 8922	[illegible]	14-0	14-0	15-0	18-0	18-0	14-0	13-0	13-0	21-0	21-0	21-0	21-0	17-0	17-0
NEW JERSEY	NJ DOT	[illegible]										23-0	23-0	23-0		
NEW MEXICO	ORDER 2202	1987	14-0	14-0	15-0	17-0	20-0	14-0	13-0		22-6	22-0	22-6	23-0	17-0	18-0
NEW YORK	NY POLICY	[illegible]	13-6	13-6	15-0	18-0	19-0	13-6	13-6	13-6	22-0	22-0	22-0	22-0	18-0	18-0
NORTH CAROLINA	NOTE 1															
NORTH DAKOTA	Sect 49-10 1-12	[illegible]									21-0	21-0	21-0	21-0	21-0	21-0
OHIO	ORDER 4963 42-43	[illegible]	14-0	14-0	15-0	15-0	18-0	14-0			21-0	21-0	21-0	21-0	21-0	21-0
OKLAHOMA	ORDER 33847	[illegible]	14-0	14-0	14-0	17-0	20-0	14-0	13-0	13-0	22-0	22-0	22-0	23-0	17-0	18-0
OREGON	ORDER 83-312	[illegible]	15-0	14-0	15-0	20-0	20-0	14-0	14-0	14-0	20-9	20-9	20-9	20-9	18-0	18-0
PENNSYLVANIA	TITLE 52 33C	[illegible]	13-6	13-6	15-0	18-0	19-0	13-6	13-6		22-0	22-0	22-0	22-0	18-0	18-0
RHODE ISLAND	DOCK 608	[illegible]	13-0	14-0	15-0	18-0	19-0	14-0	13-0	14-0	18-0	18-0	18-0	18-0	18-0	18-0
SOUTH CAROLINA	SEC 58-15-167.0	[illegible]										22-6				
SOUTH DAKOTA	ORDER F-2465	[illegible]									22-6	22-6	22-6	22-6	17-0	22-6
TENNESSEE	RULE 1220-3-1-12	[illegible]	14-0	14-0	14-0	18-0	18-0	14-0	13-0	13-0	22-0	22-0	22-0	22-0	17-0	17-0
TEXAS	CSA 6559 A-F	[illegible]									22-0	22-0	22-0	22-0	22-0	22-0
UTAH	G O 66	[illegible]	14-0	14-0	15-0	17-0	20-0	14-0	11-6	11-6	22-6	22-6	23-0	22-6	18-0	18-0
VERMONT	TITLE 30	[illegible]										22-0	22-0			
VIRGINIA	NOTE 1															
WASHINGTON	CHPTR 480-60	[illegible]	14-0	14-0	15-0	20-0	20-0	14-0	13-0	13-0	22-6	22-6	22-6	22-6	18-0	18-0
WEST VIRGINIA	G.O No 188.3	[illegible]	14-0	14-0	15-0	15-0	18-0	14-0	13-0	13-0	22-0	22-0	22-0	22-0	22-0	18-0
WISCONSIN	[illegible]	[illegible]	14-0	14-0	14-0	14-0	14-0	14-0	14-0	14-0	23-0	23-0	23-0	23-0	23-0	23-0
WYOMING	CHPTR XIII	[illegible]	14-0	14-0	15-0	17-0	20-0	14-0	13-0	13-0	22-6	22-0	23-6	23-0	17-0	18-0

HORIZONTAL (cols 18–41) — PLATFORMS (cols 24–29: A–F) · SIGNALS (cols 31–34)

1	18 GENERAL	19 THRU BRIDGES	20 HIGHWAY BRIDGES	21 TUNNELS	22 BUILDING DOORS	23 IN BUILDINGS	24 A	25 B	26 C	27 D	28 E	29 F	30 HIGH	31 LOW BETWEEN TRACKS HEIGHT	32 LOW BETWEEN TRACKS CLEARANCE	33 SWITCH BOXES ETC. HEIGHT	34 SWITCH BOXES ETC. CLEARANCE	35 POLES	36 MAIL CRANES	37 ICING DOCKS	38 ORE AND COAL DOCKS	39 CATTLE CHUTES	40 WIRES	41 PIPELINES
ALABAMA		9-0	9-0																					
ALASKA																								
ARIZONA	8-6	8-0	8-6	8-0	8-6	8-6	0-8	4-8			4-0	7-3	8-6	3-0	6-0	0-8	3-0	8-6	E	7-8	8-6	8-6		
ARKANSAS	8-6	8-0	8-6	8-0	7-0	7-0	0-8	5-0	4-0	5-9	4-0	8-0	8-6	3-0	6-0	0-4	3-0	8-6	8-6	6-6	8-6	8-6		
CALIFORNIA	8-6	8-0	8-6	8-0	8-6	8-6	0-8	4-8			4-0	7-6	8-6	3-0	6-0	0-4	3-0	8-6	8-6	7-8	8-6	8-6	[illegible]	[illegible]
CANADA	8-4¼	8-0	8-4¼	8-0	8-0	8-0	0-5	3-10			4-0	6-0	8-4¼	4-0	5-8	0-5	3-10	8-4¼	8-4¼	8-4¼	8-4¼	8-4¼	22-0	22-0
COLORADO	8-6	8-0	8-6	8-0	7-0	8-0	0-8	5-0	4-0	5-9	4-0	8-6	8-6	3-0	6-0	0-4	3-0	8-6	8-6	6-8	8-6	8-6		
CONNECTICUT	8-6	8-0	8-6	8-0	8-0	8-0	0-8	5-2			4-0	8-6	8-6	3-0	6-0	0-4	3-0	8-6	6-5	8-6	8-0	8-6		
DELAWARE																								
DIST/COLUMBIA	8-6	8-0	8-6	8-0	8-0	8-0	0-8	4-8	4-0	5-7			8-8	3-0	6-0	0-4	3-0	10-0	8-6	8-6	8-6	8-6		
FLORIDA	8-0	8-0	8-0	8-0	8-0	8-0	0-8	4-8	CFH	5-9			8-0	3-0	6-0	0-4	3-0	10-0	8-6	7-8	8-6	8-6		
GEORGIA																								
IDAHO	8-6	8-0	8-6	8-0	8-6	8-6	0-8	4-8	4-0	7-3	4-6	8-6	8-0	3-0	6-0	0-4	3-0	8-6	8-6	6-8	8-6	8-0	NOTE 24	
ILLINOIS	8-0	8-0	8-0	8-0	7-0	8-0	0-4	4-6	0-8	5-1	CFH	8-2	8-6	2-10	6-0			8-0	8-0	8-0	8-0	8-0		
INDIANA	8-0	8-0	8-0	8-0	8-0	8-0							8-0					8-0	8-0		8-0	8-0	8-0	8-0
IOWA																								
KANSAS	8-6	8-0	8-6	8-0	7-0	7-0	0-8	5-0	4-0	6-2	4-0	8-6	8-6	3-0	6-0	0-4	3-0	8-6	8-6	6-8	8-6	8-6		
KENTUCKY																								
LOUISIANA																								
MAINE	8-0	8-0	8-0	8-0	7-0	8-0	0-12	5-4	4-0	5-8			8-0	3-2	5-5	0-4	3-0	8-0	6-1	6-0	8-0	8-0		
MARYLAND	8-6	8-0	8-6	8-0	8-0	8-0	0-8	5-1	4-0	5-7	4-0	8-6	8-6	3-0	6-0	0-4	3-0	8-6	6-5	7-8	E	8-6		
MASSACHUSETTS	8-6	8-6	8-6	8-6	8-6					5-9			8-6	3-0	8-6			8-6	8-6	6-6	8-6	8-6		
MICHIGAN	8-6	E	E	8-6	8-6	8-6							E	E	E	E	E	8-6	E	E	E	E		
MINNESOTA	8-6	8-6	8-6	8-6	8-6	8-6							8-6					8-6	8-6	8-6	8-6	8-6		
MISSISSIPPI																								
MISSOURI	8-6	8-0	8-6	8-6	8-6	8-6	0-4	4-6	0-8	5-1	CFH	5-9	8-6	4-0	5-0 / 8-0	0-3	4-2	8-6	6-5	8-6	8-6	8-6		
MONTANA	8-6	8-0	8-6	8-0	8-6	8-6	0-8	4-8	4-0	5-8	4-0	8-6	8-0	3-0	6-0	0-4	3-0	8-6	8-6	5-9	8-6	8-6	NOTE 24	
NEBRASKA	8-6	8-0	8-6	8-0	7-0	7-0	0-8	5-0	4-0	5-9	4-0	8-6	8-6	3-0	6-0	0-4	3-0	8-6	8-6	6-6	8-6	8-6	NOTE 24	22-6
NEVADA	8-6	8-0	8-6	8-0	8-6	8-6	0-8	4-8	4-0	7-3	4-6	8-0	8-6	3-0	6-0	0-4	3-0	8-6	E	7-8	8-6	8-6		22-0
NEW HAMPSHIRE	8-0	8-0	8-0	8-0	7-0	8-0	0-12	5-4	4-0	5-9			8-6	3-2	5-6	0-4	3-0	8-6	6-1	8-0	6-0	8-6		
NEW JERSEY																								
NEW MEXICO	8-6	8-0	8-6	8-0	7-0	7-0	0-8	4-8	4-0	5-9	4-0	8-0	8-6	3-0	6-0	0-4	3-0	8-6	8-6	6-8	8-6	8-6	[illegible]	[illegible]
NEW YORK	8-6	8-0	8-6	8-0	8-0	8-0	0-8	5-1	4-0	5-7	4-0	8-6	8-6	3-0	6-0	0-4	3-0	8-6	6-5	8-0	E	8-0		
NORTH CAROLINA																								
NORTH DAKOTA	8-0	8-0	8-0	8-0	8-0	8-0					4-0	8-0	8-0					8-0	8-0	8-0	8-0	8-0		
OHIO	8-0	8-0	8-0	8-0	8-0	8-0							8-0					8-0	8-0	8-0	8-0	8-0		
OKLAHOMA	8-6	8-0	8-6	8-0	7-0	7-0	0-8	5-0	4-0	5-8	4-0	8-0	8-6	3-0	6-0	0-4	3-0	8-6	8-6	6-6	8-6	8-6		
OREGON	8-6	8-0	8-6	8-0	8-6	8-6	0-8	4-8			4-0	7-3	8-6	3-0	6-0	0-4	3-0	8-6	8-6	7-6	8-6	8-6	[illegible]	[illegible]
PENNSYLVANIA	12-0	8-0	12-0	8-0	8-0	8-0	0-8	5-1	4-0	5-7	4-0	8-6	12-0	3-0	6-0	0-4	3-0	12-0	6-5	8-0	E	8-6		
RHODE ISLAND	8-6	8-0	8-6	8-0	8-0	8-0	0-8	5-2			4-0	8-6	8-6	3-0	6-0	0-4	3-0	8-6	6-5	8-6	8-0	8-6		
SOUTH CAROLINA		8-0																						
SOUTH DAKOTA	8-6	8-0	8-0	8-6	7-0	8-6	0-8	4-8	CFH	5-9			8-0			0-4	3-0	8-6	8-6	8-6	8-6	8-6	NOTE 24	
TENNESSEE	8-0	8-0	8-0	8-0	8-0	8-0	0-8	4-8	4-0	5-9	4-0	7-8	8-0		6-6	0-4	3-0	8-0	6-5	7-0	8-0	8-0		
TEXAS	8-6	7-6	8-6	8-6	8-6	8-6	0-12	4-6					8-6	2-6	5-6	0-6	4-0	8-6	E	7-0	8-6	8-6	22-0	22-0
UTAH	8-6	8-0	8-6	8-0	8-6	8-6	0-8	4-8	1-6	5-8	4-0	7-3	8-6	3-0	6-0	0-4	3-0	8-6	E	7-8	8-6	8-6		22-6
VERMONT	8-0	8-0	8-0	8-0	8-0	8-0	0-8	8-0												E				
VIRGINIA																								
WASHINGTON	8-6	8-0	8-8	8-0	8-6	8-6	0-8	4-8	4-0	7-3	4-6	8-6	8-0	3-0	6-0	0-4	3-0	8-6	8-6	7-3	8-6	8-6		
WEST VIRGINIA	8-0	8-0	8-0	8-0	8-0	8-0	0-8	5-0	4-0	6-4			8-0	3-0	6-0	0-4	3-0	10-0	6-6	7-6	8-0	8-0		
WISCONSIN	8-6	E	8-6	8-6	8-6	8-6	[illegible]	[illegible]	1-9	8-0	5-0	6-4	8-6	E	E	E	E	12-0	E	8-6	8-6	8-6		
WYOMING	8-6	8-0	8-6	8-0	7-0	7-0	0-8	5-0	4-0	5-9	4-0	8-6	8-6	3-0	6-0	0-4	3-0	8-6	8-6	6-8	8-6	8-6	NOTE 24	22-6

DIMENSIONS:

ARE SHOWN IN FEET AND INCHES. THEY ARE FOR TANGENT TRACK AND MUST LAWS SPECIFY INCREASES FOR CURVED AND SUPERELEVATED TRACK

VERTICAL:

MEASURED FROM TOP OF RAIL

HORIZONTAL:

MEASURED FROM CENTER LINE OF TRACK. APPLIES TO NEW CONSTRUCTION, SOME RECONSTRUCTION AND SOME EXTENSIONS. SOME CAN BE VERIFIED UPON APPROVAL OF APPLICATION BY GOVERNING BODY. ARE BASED ON MAXIMUM CAR SIZE FOR ARIZONA, CALIFORNIA, IDAHO, MINNESOTA, MONTANA, NEVADA, NORTH DAKOTA, OREGON, WASHINGTON AND WEST VIRGINIA. ALL ARE MINIMUM EXCEPT COLUMNS 24 26 28 31 AND 33 WHICH ARE MAXIMUM.

HAWAII OMITTED — NO COMMON CARRIER RAILROADS
C = CONDITIONAL – SEE SPECIFIC REGULATION
H = HEIGHT OF CARS GOVERNS
C F H = CAR FLOOR HEIGHT
E = EXEMPT

COLUMNS:

2 SHOWS BASIC REGULATIONS
3 SHOWS EFFECTIVE DATE OR DATE OF LATEST AMENDMENT
7 & 8 APPLY TO HAND AND MECHANICALLY OPERATED SWITCHES EXCEPT AS NOTED
12 & 18 PREVAIL FOR ALL ITEMS NOT OTHERWISE PROVIDED FOR
13 & 19 BRIDGES SUPPORTING TRACKS
14 & 20 BRIDGES SPANNING TRACKS
24 & 25 PASSENGER PLATFORMS
26 & 27 FREIGHT PLATFORMS ON SIDE TRACKS AS NOTED
28 & 29 FREIGHT PLATFORMS ON SIDE TRACKS. STEPPED PLATFORMS ARE NOT GENERALLY ALLOWED
35 OTHER THAN TROLLEY CONTACT POLES
36 TO CENTER OF STAND EXCEPT AS NOTED
37 APPLIES TO BOTH SUPPORTS AND PLATFORMS EXCEPT AS NOTED

NOTES:

1 NO CURRENT REGULATION – CHECK WITH RAILROAD INVOLVED
2 LESSER CLEARANCES NOT PERMITTED IN QUADRANTS
3 ENGINE HOUSES AND SHOP BUILDINGS EXEMPT, OR PERMITTED LESSER DIMENSIONS
4 ONLY IF TRACKS END WITHIN BUILDING
5 MAY BE REDUCED TO 5-9 IF 8-3 (8-6 FOR WYOMING) PROVIDED ON OPPOSITE SIDE.
6 MAY BE REDUCED TO 5-9 IF 8-0 (8-6 FOR NE & WY) PROVIDED ON OPPOSITE SIDE
7 ONLY IF 8-0 (7-3 FOR MT & WY 8-6 FOR WA, MA & SD) PROVIDED ON OPPOSITE SIDE
8 MAY BE LESS ON ONE SIDE IF FULL GENERAL (COL 18) PROVIDED ON OPPOSITE SIDE
9 PASSENGER PLATFORMS ONLY
10 MAY BE 8-0 AT 4-6 FOR REFRIGERATOR CAR PLATFORMS ONLY
11. TO ENDS OF ARMS IN OPERATING POSITION
12 FOR PLATFORMS AT CAR EAVE HEIGHT – SUPPORTS TO BE 8-0 (8-6 FOR TEXAS)
13 FOR MAIN TRACKS – ALL OTHER 22-0
14 8-6 FOR MAIN AND PASSING TRACKS
15 FOR FREIGHT TRACKS – CAN BE 13-0 FOR PASSENGER TRACKS
16 FOR HAND OPERATED SWITCHES – MAY BE 15-0 FOR MECHANICALLY OPERATED SWITCHES
17. FOR HAND OPERATED SWITCHES – MAY BE 17-0 FOR MECHANICALLY OPERATED SWITCHES
18 PARALLEL LEAD TRACKS 18-0
19 MAY BE REDUCED TO 6-2 IF 8-3 PROVIDED ON OPPOSITE SIDE
20 MAY BE REDUCED TO 6-2 IF 8-0 PROVIDED ON OPPOSITE SIDE
21 MUST HAVE ADDITIONAL SIDE CLEARANCE OF ONE INCH PER 30 MINUTES OF TRACK CURVATURE
22 STATE REGULATIONS CONFORM TO A.R.E.A. STANDARDS. SEE CHAPTERS 14 & 28 FOR SPECIFICS
23 ONLY IF 8-6" CLEARANCE AND 14-0" TRACK CENTERS PROVIDED ON OPPOSITE SIDE
24 STATE REGULATIONS CONFORM TO NATIONAL ELECTRICAL SAFETY CODE

11-15-85

RESULTS OF PHASE II HEAVY AXLE LOAD TESTS AT FAST

By: David Read* and Semih Kalay**

Introduction

The Heavy Axle Load (HAL) program at FAST has, since 1988, investigated the effects on the track structure of operating 315,000-pound Gross Vehicle Weight (GVW) cars with 39-ton axle loads. Full-scale test operations are performed on the 2.7-mile High Tonnage Loop (HTL) at the Transportation Technology Center, Pueblo, Colorado. Between 100 and 150 million gross tons (MGT) of traffic is generated annually with a 76-car train. Experimental data obtained under controlled operating conditions is utilized to define the economic and safety ramifications of 39-ton axle loads and contributes to the overall AAR goal of optimized axle loads for North American freight operations.

The performance of conventional track materials under HAL traffic was measured in Phase I of the program between 1988 and 1990. To the extent possible, Phase I tests were designed to allow direct comparison with 263,000-pound GVW performance recorded earlier at FAST. Data obtained during this initial 160 MGT phase of operations suggested several areas of concern, including thermite welds, special trackwork, track with marginal support and rail fatigue. The data was also incorporated into the Phase I HAL economic analysis which predicted an increase in track maintenance effort by approximately 23-percent with 315K traffic. The results from Phase I implied that the use of premium track components could improve the economics of 315K operations. The next phase of the HAL program, therefore, was designed and implemented to determine the benefit of premium track components, especially in those areas where Phase I showed potential improvement was needed. Additional tonnage was also needed to better define long term fatigue related HAL effects. At its completion in early 1995, 300 MGT of tonnage had been generated during Phase II, for a total of 460 MGT of HAL traffic overall.

Phases I and II were conducted with the FAST train equipped with standard 3-piece freight car trucks. The current operation (Phase III), which began in late 1995, continues to define the economic potential of HAL operations by evaluating the benefits of improved suspension trucks on premium track components.

The following is a synopsis of results produced during Phase II of the FAST/HAL operation.

FAST Train on HTL

*Sr. Engineer I, AAR Transportation Technology Center
**Chief Tech. Officer, AAR Transportation Technology Center

Rail Wear

Rails from manufacturers worldwide were tested on the 5-degree curve of Section 07 of the HTL (Figure 1). The rail in Section 07 was not directly lubricated, but received a slight amount of lubrication carried from the other curves. Rails tested during Phase II included several premium head-hardened rails ranging in hardness from 320 to 370 Brinell (Bhn), as well as 300 Bhn standard rail installed as the control.

Results indicated that:

- The forecasted wear life of 300 Bhn standard rail on a 5-degree curve, as defined by the amount of tonnage a rail can be exposed to before accumulating ½ inch of wear at the location of measurement, under 39-ton axle loads was 272 MGT. The forecasted wear life of premium head hardened rail ranged from 338 MGT to 588 MGT (Figure 2).
- The gage face wear rate of rail with hardness of 320 Bhn or higher, decreased at a linear rate with increasing rail hardness. This analysis predicted the wear rate of 370 Bhn rail at 0.8 inches/1,000 MGT and 1.5 inches/1,000 MGT for 320 Bhn rail. The standard rail wear rate was about 1.8 inches/1,000 MGT.
- The vertical wear of the standard rail was 1.4 to 2.8 times higher than the vertical wear rate of the head hardened rail. The higher rate of vertical wear measured on standard rail was due to plastic deformation, or "crushing," in response to vertical forces. Significant deformation was not observed on the head hardened rail.
- The standard rail was also the only rail to corrugate. Corrugations were observed in the 300 Bhn control rail at 15 MGT with wavelengths from 12 inches to 18 inches. By 85 MGT, standard rail corrugations were as deep as 0.11 inch. None of the head hardened rails developed visible corrugations during the same 85 MGT test period.

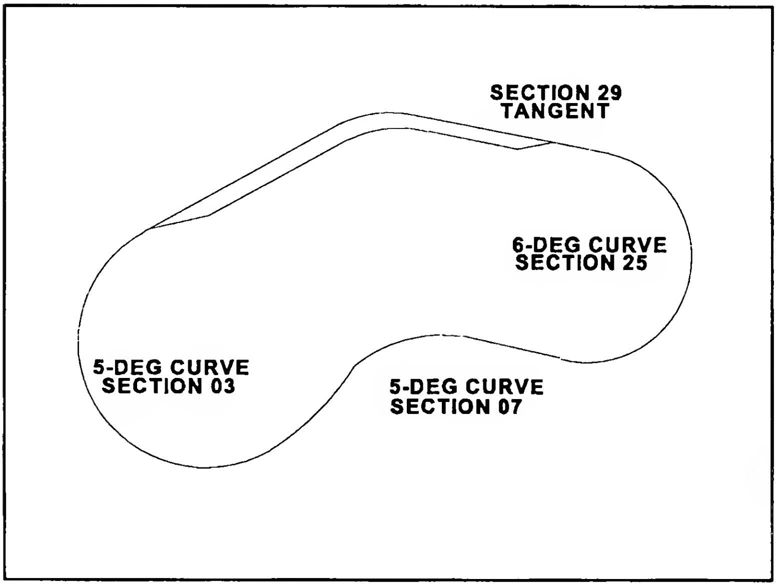


Figure 1. Primary HTL Test Zones.

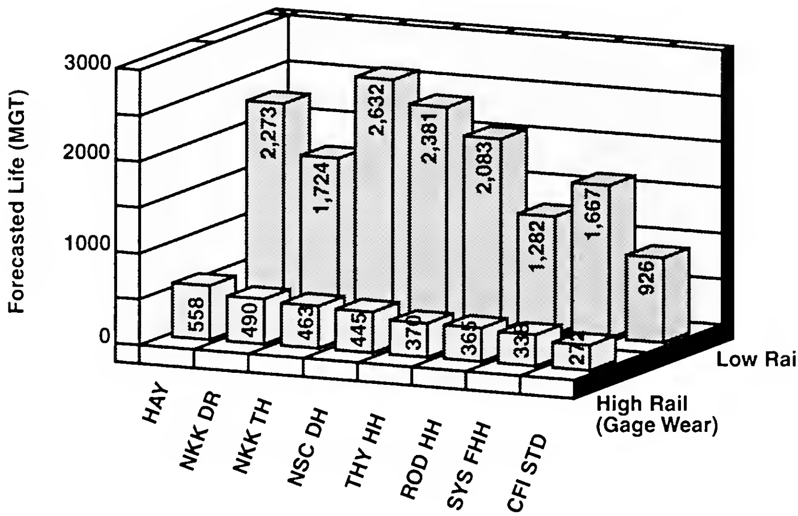


Figure 2. Forecasted Rail Life.

- Rail lubrication (gage face coefficient of friction 0.15–0.20) increased wear life by as much as 14 times that of lightly lubricated rail (gage face coefficient of friction 0.25–0.35).

Rail Profile Grinding

Two rail profile grinding tests were implemented in Phase II to replicate current industry grinding practices and help determine the effects of grinding on rail fatigue and wear. The tests were performed in the 6-degree curve of Section 25 and the 5-degree curve of Section 03. Both curves were well lubricated with the typical coefficient of friction on the high rail top between 0.25–0.30, high rail gage face between 0.15–0.20, and top of the low rail between 0.30–0.40.

Rail Profile Grind Test #1

The first test was performed over a 180 MGT period using 300 Bhn standard rail ingot cast 136-10 RE section and continuously cast 136-10 RE 300 Bhn standard rail section. Test zones included control zones with no grinding, a 2-point contact profile zone ground at 12.5 MGT intervals, 2-point contact profile zones ground at 25.0 MGT intervals, and conformal contact profile zones ground at 25.0 MGT intervals.

Two-point contact was produced by establishing and maintaining a profile that produced a 0.025 inches of relief (from a naturally worn rail) on the high rail gage corner. The conformal profile was produced by establishing and maintaining (with the indicated metal removal rate) a naturally worn FAST high rail profile.

Continuously cast 136-10 rails produced no defects throughout the test, regardless of the grinding maintenance practice selected. Metallurgy lab results showed this rail to be cleaner and slightly harder than the ingot cast rails. Since this rail was maintained the same as the 136 RE ingot cast rails, its resistance to fatigue was attributed to improved steel characteristics.

Beginning at about 100 MGT, the ingot cast rails produced defects in all zones except the 2-point contact zone maintained at 12.5 MGT intervals. This zone remained defect free throughout the test, although it did have the highest gage face wear rate. Shelling rates of the ingot cast rails were highest in the control zones and in the conformal profile zones. However, the detail fracture rates were similar in the control, conformal profile, and 2-point contact 25 MGT interval zones. Two-point contact profiles ground at 25 MGT intervals reduced the shelling rate but not the detail fracture rate as a higher percentage of shells turned into detail fractures.

Rail Profile Grind Test #2

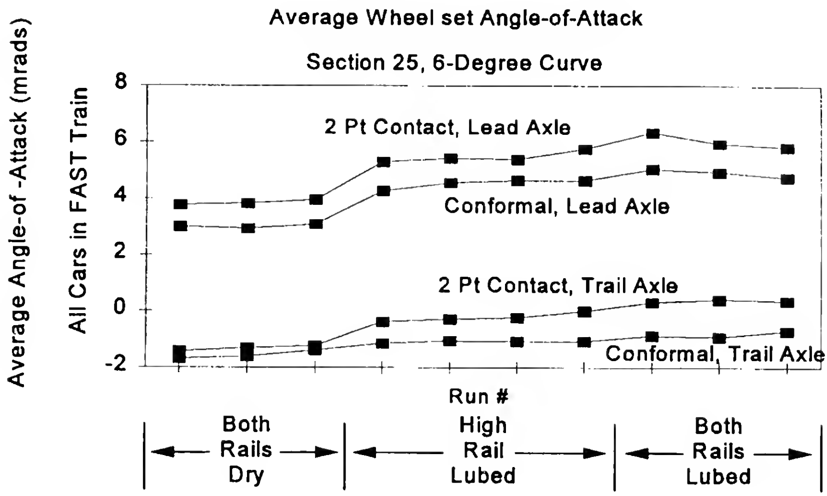
During the second profile grind test, the curves were divided into zones representing specific grinding practices relating to the amount of metal removed from the gage corner and the tonnage interval at which the grinding is performed. Control zones and a dry wear zone were also established. In contrast to the first test, only head hardened rail was used in the second test which accumulated 100 MGT at the end of Phase II operations.

After 100 MGT, the second test produced the following results:

- No rail defects originated in the head hardened rail during the 100 MGT test period. It is conjectured that at least 400 MGT will be needed for the premium rail to begin developing internal fatigue defects and it will be several years before fatigue results for these rails will be available.
- Rail wear measurements showed a higher wear rate on rails with gage corner relief than on rails where grinding was not performed. The wear rates measured in the Section 25 grind zones were all similar (approximately 0.080 inches after 100 MGT) even though the grinding intervals varied from 12.5 MGT in the aggressive zone to 75 MGT in the passive zone. Rails that were not profiled (control zones and the dry wear zone), showed wear of less than 0.010 inches after 100 MGT.
- Measurements taken in Section 25 showed that 2-point contact caused an increase in the wheel set angle-of-attack and lateral forces. The effect of grinding on vehicle curving behavior was measured under the FAST consist with wayside rail force instrumentation. Vertical and lateral rail force data was collected initially on a worn rail with conformal wheel/rail contact. The gage corner of the same rail was then relieved 0.040 inch by grinding and curving data collected again. The average angle-of-attack and average low rail lateral force are shown in Figure 3. The data demonstrates that high rail gage corner relief generated higher wheel set angle-of-attack under all lubrication conditions. Lateral forces, which are in part a function of low rail lubrication conditions, were highest when the low rail was dry. Two-point contact produced generally higher lateral forces when both rails were dry and when the high rail was lubricated and the top of the low rail was dry. Lateral forces were lowest and when both rails were lubricated. Lubricating both rails brought forces down and is the approach often taken in revenue service to control forces, especially where two point contact grinding is used.
- Spalling of the low rail running surface was not evident in the non-grind control zones or the dry wear zone after 100 MGT. Significant low rail spalling developed in two zones by 50 MGT: the wood tie zone in Section 3 and the Section 25 passive grind zone. Both zones were similar in that they received gage corner relieve profile grinding and were located on cut spike track with wide gage.

Thermite Weld Performance

During Phase I, the average life of high-rail thermite welds was approximately 62 MGT, with 144 MGT maximum life and the average life on the low rail was 114 MGT. Most welds failed from shelling and horizontal web cracks. As a result of the Phase I performance, the AAR and the weld manufacturers set out to improve the reliability of welds under 39-ton axle loads. AAR implemented extensive welder training and the manufacturers concentrated on changes to procedures and designs, including a new mold design aimed at reducing failures from web cracking.



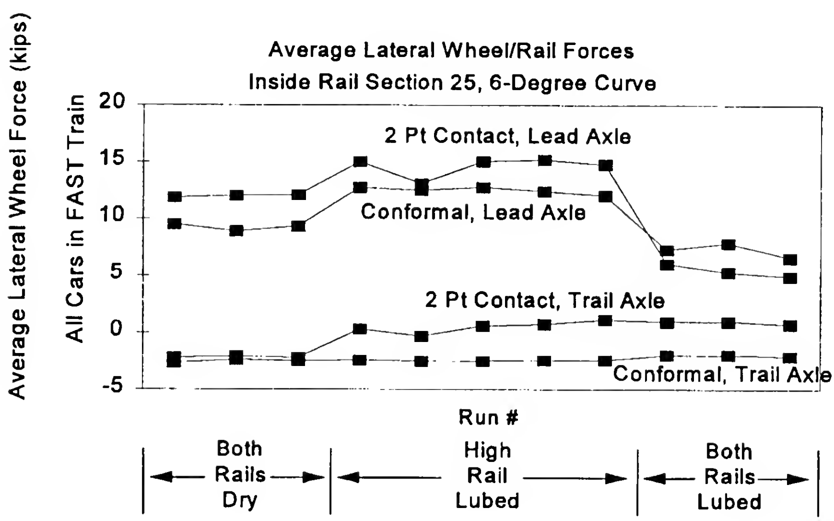


Figure 3. Effect of Rail Profile and Lubrication on Angle-of-Attack and Lateral Forces Measured Under Consecutive Passes of the FAST Consist.

In Phase II, a new weld test was implemented with combinations of standard and premium chemistries and/or standard or modified mold designs installed in a 5-degree curve test section of the HTL. Seventy-one thermite welds were installed, including 36 standard welds (16 on the low rail and 20 on the high rail) and 35 premium welds (18 on the low rail and 17 on the high rail). Results of Phase II thermite weld testing after 225 MGT include:

- Compared to Phase I, the life of standard and non-experimental premium welds increased as shown in Figure 4.

- Premium welds displayed a higher resistance to batter than did standard welds. The average batter of premium welds was about 25-percent less than the standard weld average.
- Improvements in current mold designs provide a smoother web which reduces the grinding maintenance required at the web of the weld.
- Welder training and adherence to procedures is a critical aspect of weld performance under 39-ton axle loads.

Turnouts/Frogs/Crossing Diamonds

Three turnouts were tested to monitor the performance of No. 20 or equivalent turnouts under 39-ton axle loads: (1) a standard component No. 20 turnout with AREA geometry, installed at the beginning of the program and removed after about 100 MGT during Phase I, (2) a premium component No. 20 AREA geometry, with heat treated rail, elastic rail fasteners, premium Rail Bound Manganese (RBM) frog, and thick-web switch points turnout which remains in track after 350 MGT, and (3) a No. 18½ tangential geometry turnout with swing nose frog, asymmetrical switch points and concrete ties.

Five rigid No. 20 frogs and two No. 10 spring rail frogs were tested in addition to the turnouts. The frogs included an alloy (Vario) European design fixed point frog (UIC 60 rail section), two high integrity AREA design RBM frogs, a European design fixed point manganese frog (UIC 60 rail section), a standard non-hardened RAM frog, a standard component spring rail frog, and a premium component spring rail frog.

Test results indicated that steel hardness and quality were the most important variables relating to turnout and frog performance under 39-ton axle loads. Component design, including turnout geometric design, had far less impact than did the material quality on the behavior and longevity of the special trackwork in test. Specific results include:

- The AREA geometry No. 20 turnout with premium components lasted 350 MGT of 39-ton axle load operations as opposed to standard component turnout which was removed after 100 MGT.
- The maximum lateral force measured through the switch of the AREA geometry turnout was 22,000 lb as compared to 13,000 lb in the tangential turnout. The maximum vertical force at

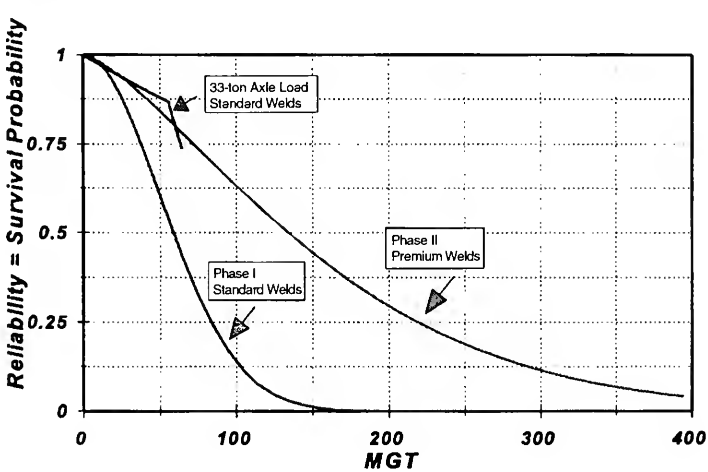


Figure 4. Comparison of Phase I/II High Rail Termite Weld Life

the RBM frog in the AREA turnout was 90,000 lb and slightly less than 60,000 lb at the swing-nose frog in the tangential turnout. The benefit of the tangential/swing-nose design and lower forces, however, was not realized until the turnout was rebuilt with premium rail. When installed, the hardness of the rail in the tangential geometry turnout was less than 300 Bhn with selected locations along the switch point and frog heat treated to approximately 320 Bhn. The rail rapidly battered at the edges of the heat treatment locations, and spalled and corrugated throughout the turnout. The soft rail required almost constant maintenance and was replaced after 130 MGT with head hardened rail.

- The service life of manganese castings in No. 20 AREA design RBM frogs varied from 70 MGT for a non-hardened, thin wall design, to over 360 MGT for an explosive depth hardened (EDH), high integrity casting. The other two high integrity RBM frogs were removed from service at 203 MGT and 172 MGT. The European design manganese frog suffered an early failure at 79 MGT, due to an inclusion in the casting. The life of the Vario frog was 277 MGT, although the frog was constructed with UIC 60 kg/m rail section which is somewhat smaller than the AREA 132/136 rail sections used on the other frogs.
- Both spring rail frogs had relatively short lives with the premium frog removed at 44 MGT due to a horizontal crack in the point rail and the standard frog removed at 69 MGT with a vertical split head in the point rail.

Five crossing diamonds were also tested under 39-ton axle loads. An 89-degree standard component manganese insert crossing was removed from track after 1.9 MGT due to excessive plastic deformation of the manganese steel at the gap. The castings were then explosion hardened and reversed and the wing rails replaced with premium rail. The rebuilt 89-degree crossing remained in track for 15 MGT. A 62-degree three-rail bolted crossing with about 14 MGT of revenue service traffic was installed on the HTL after the rail ends at the crossing gaps had been repaired. It survived an additional 4.6 MGT. The running rails of this crossing were then replaced with premium rail. The crossing was reinstalled and lasted 29.4 MGT. A 76-degree premium component solid manganese crossing survived 15.9 MGT but required extensive weld repairs at 10.8 MGT. The crossing diamond history is summarized in Table 1.

Instrumented wheel sets were used to measure 39-ton axle load impact forces in the crossings at various speeds. The impact forces generally increase linearly with speed through 40 mph and the average impact force was between two and three times the static load at 40 mph depending on the crossing angle (Figure 5).

Crossing diamond tests conducted at FAST, seem to indicate that current frog designs and materials will not survive long under 39-ton axle loads. The AAR is investigating advanced designs and materials as part of its Special Trackwork Strategic Research Initiative.

Table 1. Summary of Crossing Diamond Performance.

Crossing	Life (MGT)	Reason for Removal
89-Degree Manganese Insert	1.9	Rail Batter and Corrugation
62-Degree Three-Rail Bolted	4.6	Battered and Cracked Rail End
76-Degree Solid Manganese	15.9	Cracked Casting
Rebuilt 62-Degree Three-Rail Bolted	29.4	Head-Web Crack in Rail End
Rebuilt 89-Degree Manganese Insert	15.2	Broken Wing Rail

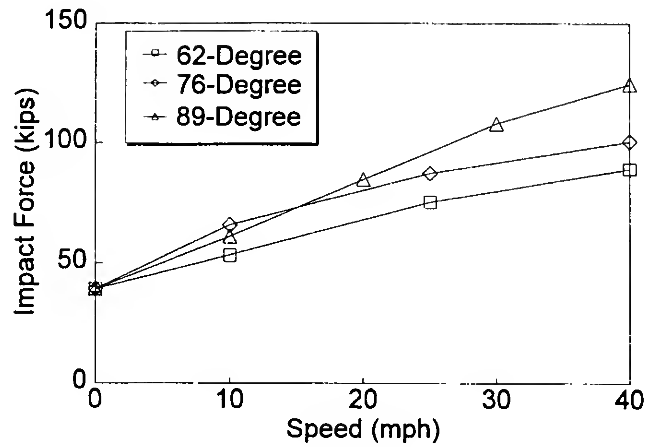


Figure 5. Average Crossing Impact Forces under HAL Cars.

Wood Tie and Fastener Performance

Wood ties with assorted rail fasteners have been under test throughout the HAL program. During Phase II, much of the testing was concentrated in the 6-degree curve of Section 25. The curving forces and L/V ratios tend to be the highest in this curve, therefore, it represents the most severe operating environment on the HTL. In general, the performance of ties of various wood species equipped with cut spike and elastic (Pandrol) rail fasteners under 39-ton axle loads was quantified. Gage retention as a function of applied tonnage was the primary indicator of tie/fastener performance. Test results were as follows:

- Of the ties tested in Section 25 with cut spikes, which included oak, Douglas fir, hem-fir, southern yellow pine, and red maple, oak was the only species to accumulate over 400 MGT without requiring re-gaging. Maximum gage widening (ignoring rail wear) of oak ties after 460 MGT was about 0.75 inches. Gage widening of hem-fir and southern yellow pine exceeded I inch (FRA class 4 track safety standard) and required re-gaging at 200 MGT. Douglas fir ties required re-gaging between 360 MGT and 375 MGT and the red maple ties were ready for re-gaging when removed from track after 360 MGT.

Use of the Pandrol fastening system with screw spikes significantly improved the gage retention capability of the hem-fir, southern yellow pine and Douglas fir ties. Installed when the ties were re-gaged, the rate of gage widening with the Pandrol system was about half that experienced with cut spikes when the ties were new. There was a tendency, however, for some coach screws to back out of the ties under traffic. Threaded coil inserts were installed in the coach screw holes to remedy the problem, but were not effective if the tie was split at the hole. The performance of the softwood ties in Section 25 suggests that changing from cut spikes to an elastic fastening system with screw spikes will improve the existing gage restraint, but the long-term effectiveness of the elastic fastening system will depend on the tie condition.

Concrete Ties

Concrete ties have been under test in the 5-degree curve of Section 03 since the start of the HAL program in 1988. Other than some tie center cracking that occurred early in the program—all of the cracked ties have remained in track—There has been no indication of concrete tie problems during 460 MGT of 39-ton axle load operation.

In addition to concrete tie structural performance, the issue of rail seat abrasion has been also studied. A number of materials and components designed to resist or prevent abrasion have been evaluated. Results suggest there are specific pad types which can reduce or prevent abrasion and that repair techniques which appear to survive HAL traffic are also available.

Many abrasion resistant tie pads may not provide a permanent solution. Instead, they may require replacement on a periodic basis over the life of the tie. Repairing rail seats and replacing tie pads between programmed rail replacement cycles can be a significant cost item. For this reason, the target life cycle between tie pad replacement should be the same as the rail at a given location.

The following pad materials seem to offer promise in preventing abrasion:

- Dual durometer materials (hard and soft materials bonded to a single pad).
- Sandwich materials (multiple products such as a sealant, steel plate and resilient material on one pad).
- Convex rail seats.

Low Track Modulus/Load Path Evaluation

The soil at the HTL provides an excellent subgrade for railroad track support. Consequently, the track modulus and stiffness of the HTL is believed to be higher than much of revenue track. Although the relationship between track modulus and track performance has not been firmly established, it is likely that much track maintenance expense is related to deficiencies in the track substructure which allow large rail deflections under load. To gain as complete a determination as possible of track performance under 39-ton axle loads, a low track modulus (LTM) test zone was installed in Section 29 by replacing the existing soil with a low strength clay. The purpose of the LTM zone was to simulate lower-end, but not worse case, mainline track support conditions. A track modulus of 2,000 lb/in/in was used as the target value for the LTM design and construction. A control zone with modulus of approximately 5,000 lb/in/in was also established.

In addition to quantifying track geometry degradation, the LTM and control zones were equipped with instrumentation to measure vertical load path characteristics. A measurement cell was installed in both zones to collect vertical rail force, vertical rail seat force, and ballast/subgrade pressure data. Load path data was collected under a consist of equal numbers of 33- and 39-ton axle load vehicles.

Results of the Load Path Evaluation and LTM experiments are as follows:

- Track modulus less than 2,000 lb/in/in is not recommended for sustained operations of 39- ton axle loads. Modulus of between 2,000 lb/in/in and 2,500 lb/in/in should be considered marginal for HAL operations, with frequent maintenance cycles to be expected. The LTM zone required surfacing at 12, 28, 37, 48, and 60 MGT due to cross-level deviations exceeding 1.5 inches. Geometry degradation in the control zone was not significant during the same 60 MGT period.
- Vertical subgrade pressures measured under 39-ton axle load vehicles were in the range of 10 psi to 20 psi in both the LTM and the control zones. The difference in subgrade stiffness between the two zones did not have a significant effect on subgrade pressures, therefore, pressures in excess of 10 psi should be expected under HAL traffic.
- The effects of 39-ton axle load rail impact forces, as generated by a wheel flat or battered weld, on subgrade pressure was measured by grinding a flat spot into the rail head directly over a subgrade pressure cell. Subgrade pressures increased 50 to 60 percent under impact forces generated by a 0.080-inch flat spot in the running surface of the rail head when compared to subgrade pressures under a smooth running surface.
- Dynamic vertical wheel forces from a standard 3-piece, variable damped truck increased linearly with track profile roughness. The rate at which the forces increase with profile degradation will serve as a baseline for the evaluation of advanced truck design performance.

Phase III

Phase II test results showed that premium track components, especially rail, frogs, thermite welds, and wood ties with elastic fasteners have a significant performance advantage over standard materials under 39-ton axle loads. The current FAST/HAL operation (Phase III) is measuring the effects of improved truck designs on premium and standard track degradation. In 1995, the FAST train was re-equipped with trucks having primary suspensions and increased warp stiffness. These trucks have improved curving characteristics and should generate lower lateral forces on curves. The primary suspensions may also have some benefits in terms of vertical force attenuation which would reduce track geometry degradation and rail surface fatigue. Preliminary Phase III data indicates that the designs have substantially reduced curving forces, resulting in much lower rates of rail wear and rail corrugation development.

BNSF'S HOBSON YARD CONSTRUCTION PROJECT

By: D. W. Ferryman*

In December of 1992, the Burlington Northern Railroad asked me to be the Project Manager of the Hobson Yard Project in Lincoln, Nebraska. At the time I knew very little about the Hobson Yard, but it sounded like a great opportunity, so I happily accepted the job. I figured that the project would be fairly basic: lay down some dirt, build a few switches and tracks, dump rock, raise, regulate and run trains. I decided it would be in my best interest to take a trip down to the existing Hobson Yard, and I was amazed at what I found. In the middle of the Classification Yard was a lake, and not by design. The pond was affectionately known as Lake Hobson. Rather than bringing a claw bar and spike maul to start construction, a fishing pole somehow seemed a more appropriate tool. Numerous tracks were buried in water and grain was piled up between the rails. I was wearing all of my personal protective equipment, with one exception. I forgot my nose plug. The smell of the fermenting grain rising off the black waters of Lake Hobson was almost unbearable. I then inquired where we would be beginning the new construction, expecting them to point at the empty farmland to the South, and they said you're looking at it. Now when they hired me, I knew they expected a lot from me, but they didn't say anything about walking on water, let alone running trains. Hobson Yard was originally built on what many would consider to be useless swamp land. What very little drainage existed was clogged up by grain and mud over the years. The track conditions in the yard were deplorable. As I walked through the yard, I counted 17 gage rods on one particular switch. They told me that it was not out of the ordinary to put cars on the ground 5 days out of the week. It was imperative that a complete yard reconstruction be done. After experimenting with several designs and discovering a Cadillac can't be built with a Volkswagen budget, a design was agreed upon and the money was approved to start the work. The Hobson Yard Expansion/Reconstruction is a 4 year, $25 million project.

Figure 1.

*Project Manager, BNSF

The Hobson Design is what we call a "Wrap-Around Yard." On one side is a Departure Yard, on the other is an Arrival Yard, and in the middle is the Classification Yard (or bowl), thus the name "Wrap-Around Yard."

The old classification yard at Hobson was nothing more than a flat switching yard, where cars had to be "kicked" and could couple at speeds around 10 MPH damaging the internal freight. The new Classification Yard consists of 32 tracks, ranging in length from 1200' to 3100'. Total capacity of the bowl is 1120 cars. On the West end of the Classification Yard is a hump. However, this hump is quite a departure from traditional humps. Much smaller, we refer to it as a mini-hump. The ascending grade coming into the hump is +0.79%. This removes any slack between the cars, allowing their knuckles to come together so the pin may be pulled while the cars are being shoved. Then the cars reach a vertical curve and begin accelerating on a descending grade of –1.9%. This steep grade is required to gain proper spacing between the cuts of cars on the lead. The car then passes through another vertical curve and enters onto a continuous grade of –.45% on its entire voyage down the lead. Although the grades on each track are slightly different, once the car has entered its designated track the grade drops to around –0.1%. At the East end of the Classification Yard is an ascending grade of around +0.4%. This is designed to keep the cars from rolling out of the bowl. These are the ideal grades for an empty box car on the 4th of July with no atmospheric disturbance, to accelerate to 6 MPH, remaining at 6 MPH down the lead and into the classification tracks, slowing to a constant speed of 4 MPH, and coming to a stop at the East end. This is where the problems arise. There are countless variables with every train car, and Lincoln, Nebraska, may have a consistently good football team, but the weather is anything but consistent. Thus the leads and the tracks need retardation. Both leads are supplied with Ultra Hydraulic retarders that keep the cars at 6 MPH down the lead. Each turnout is equipped with a speed control zone of retarders designed to keep the cars at 6 MPH throughout the turnout. At the clearance point within each of the tracks, a deceleration zone of retarders exists which slows the cars from 6 MPH down to 4 MPH. The cars then couple safely at a speed of 4 MPH or less.

In an effort to keep water away from the retarders, a drainage system was installed. The system links a series of French drains underneath the deceleration zone of each track.

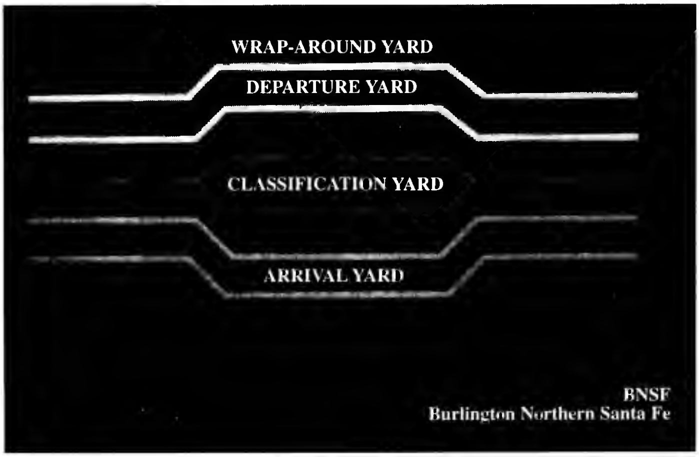


Figure 2.

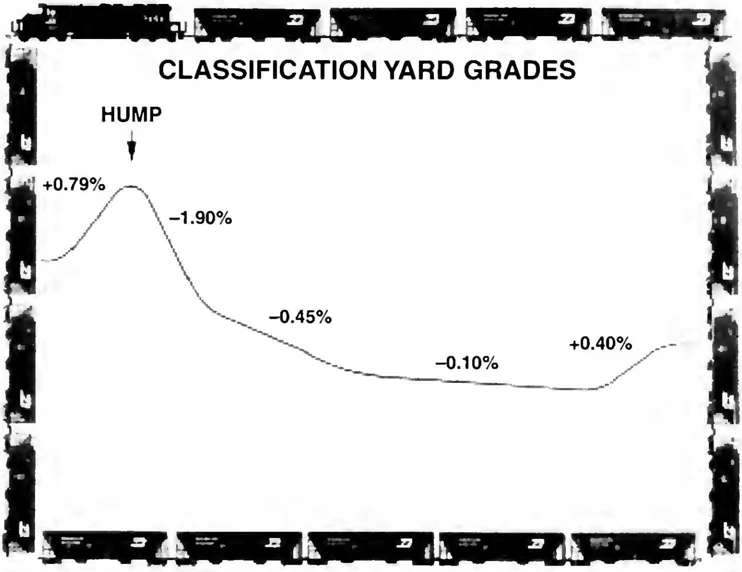


Figure 3.

The 31 Air Powered Switches on the West Leads are controlled from a computer located in the Crest Building. This computer is controlled by a Switch Foreman, who can type up to 3 routes into the computer before the cars pass over the hump. The leads are equipped with presence detectors that track the location of each cut of cars as they travel down the lead. Once each cut has entered its correct track, the switches automatically align for the next cut of cars.

Construction of the Classification yard was done in 2 stages. The South side of the bowl was constructed in 1993 & 1994, and the North side of the bowl was constructed in 1995. I quickly discovered that my biggest challenge was to construct without bringing the yard operations to a halt. This took a large amount of giving on both sides. On the construction side, we had to build a few temporary tracks and the Terminal Superintendent had to exercise a creative operating plan, since we were taking half of his bowl from him. Our initial plan was to relay each of the tracks with the P811 and shift them to their correct location. However, the experiment failed. The P811 track renewal machine is an extraordinary machine for track renewal on the Main Line, but is not prepared to dig-up old track imbedded in mud. We then resorted to the quickly conceived Plan 'B'. This required us to remove all of the old Classification Tracks. This also provided us with the opportunity to rework the sub-grade and install additional drainage. Tracks were then laid in their correct location with the Fairmont/Tamper Track Laying Machine. Yard ballast was then dumped on each of the tracks. The tracks were then raised anywhere from 1 to 2 feet and put to their exact grade.

Once the South Bowl was completed, it was time to cut over the new hump. This project required the shut-down of the existing classification operation for 24 hours. This "around the clock" project started with the removal of 4 switches. Once tracks were removed, all of the soft spots in the soil had to be cored out and built back up to a precise grade. Three prebuilt switches were then placed in their new location and 75 feet of track was built connecting the switches. Yard Ballast was then dumped on the track and all the switches and trackage were raised to their exact elevation. Once all

Figure 4.

Figure 5.

the switch machines and presence detectors were installed, the first car traveled over the hump 24 hours after the first spike was pulled.

Another challenge we were faced with in the construction of the classification yard was raising the East Lead (Trimmer End) up high enough to keep cars from rolling out of the bowl. We were shocked the first time we set a top of rail stake, showing us the elevation we needed to raise the track to provide an ascending grade of +0.4% on the East end of every track. To mark this elevation we had to tape 2 pieces of laft together. The average raise on the East end was approximately 2 feet. However, with the added drainage and the amount of rock under each of the tracks, Lake Hobson is a distant memory.

In 1994 the Departure Yard was also constructed. The Departure Yard lies North of the Classification Yard and consists of 5 Tracks with an average length of 8500 feet per track and a total capacity of 708 cars. Blocks of cars are assembled within the Classification Yard. These blocks are then pulled out either the North or South Pull Track and then pushed back into the Departure Yard by a Trimmer Locomotive. Block by block, trains are assembled within the Departure Yard. Fresh power from the diesel pit is then supplied to either end of the train depending upon which direction the train will depart. The Northern most Departure Track is called the Engine Run-around and has direct access to the Freight Pit for this purpose.

To construct these tracks, we needed to establish an area of new grade on an existing wetland. This project required us to obtain a 404 permit from the U.S. Army Corps of Engineers. Once approved we began construction. A lead was constructed at the East end of the Departure Yard as well as a series of interchanges and cross-overs allowing several options for departing 2 trains out of the yard at the same time. Construction of the departure yard dissected what was known as the old North Hump. These tracks were stubbed to make room for the new Departure tracks. Once the new grade and the existing grade were matched up, the Track Laying Machine returned and connected the new East lead into 5 of the old receiving tracks on the West end. The finished product is 5 tracks at 8500 feet.

Construction of the arrival yard, just South of the Classification Yard, begins in 1996. The Arrival Yard consists of 4 tracks with an average length of 8500 feet, and a capacity of 560 cars. Upon completion the inbound trains will be able to pull into the Arrival Yard from either direction. The power on these trains will then travel to the Diesel Pit for service. The Hump Locomotive can then travel into the West end of the Arrival Yard and pull a train back to the Hump Lead for classifying.

A good portion of the land for the Arrival Yard construction is wetland, which will require further mitigation with the U.S. Army Corp of Engineers. Along with several lead changes at the far East and West end of the Hobson Yard, this is all that remains of the track work on the Hobson Yard Project.

The project has left old trackage in several areas. We have found various uses for these tracks. The old short receiving yard on the West end provides an additional area to arrive short trains that need classifying. This Receiving Yard has direct access to the Hump. Several old tracks on the East end of the yard will be used for a Local Departure Yard. Short trains can be built from the Classification Yard into this yard and departed. The old North Hump that the Departure Yard dissected is now known as the North Yard and is used primarily for storage and Maintenance of Way cars.

Currently the old Yard Office is still being used and a new Yard Office building is under construction. The old Yard Office is foul of the North Bowl East Lead construction. A temporary lead alignment was made until the old Yard Office is removed. Once removed, grade work will be finished and the lead completed.

The new Yard Office is a facility that will centralize the dispatch of all Yard and Outbound crews to their trains via front door van service.

Currently Hobson Yard is classifying 500 to 600 cars per day. Upon completion, we estimate the classification of 900 to 1000 cars per day. The potential is for 1200 cars per day, but due to the current high volume of coal traffic through Lincoln, the cars are not available to meet this potential. Other construction is currently underway to expedite coal train traffic through the Hobson Yard.

SPRINGFIELD RAILROAD RELOCATION PROJECT

By: J. W. Moll*

After years of delays and change in direction, a major railroad relocation project that has improved safety and simplified traffic patterns in Springfield, Illinois, finally reached its destination last year.

The Capital City Railroad Relocation project moved the Southern Pacific and Norfolk Southern railroad tracks from a congested area in Southwest Springfield to a single corridor South and West of the city. The move eliminated three railroad grade crossings and greatly eliminated a fourth, all of which had a history of accidents. A narrow subway crossing was also removed, allowing a major city street to be widened and improved.

As the project's engineering consultant, Hanson Engineers Inc. was part of the team that brought this process to completion. The story of how this came to pass extends into the last century and offers a study of a process complicated by changes in the railroad industry, and changes in city growth patterns and the local economy.

Railroad Service Evolves in Springfield

The first railroad came to Springfield in 1842, five years after Abraham Lincoln. This railroad became the Great Western, then the Wabash, then the Norfolk & Western, and ultimately the Norfolk Southern. Later, the Chicago and Alton Railroad also came to Springfield, forming only the third railroad crossing in the state (Figure 1). That crossing was at Iles Junction, which figures prominently in this story. The Chicago & Alton line is now owned by the Southern Pacific.

Springfield's location contributed to its continued development as a railroad center, and by the early part of this century a network of rail lines served the city (Figure 2). The railroads were originally located in the central city area to serve the needs of the community at the time of their construction (Figure 3). As the city grew, however, there were increasing conflicts between the operations of the railroads and the function of the community.

These concerns led, as early as 1924, to efforts to eliminate ground-level crossings, with a plan that called for elevated tracks throughout the city and an extensive system of yard tracks in the central city area. This plan was never implemented.

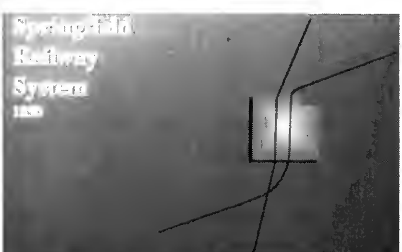


Figure 1. Springfield Railway System, 1850.

Figure 2. Springfield Railway System, 1916.

*Partner, Hanson Engineers Inc.

Figure 3. Railroad Service to Downtown Springfield.

The years leading to the 1960's saw significant changes in both rail and auto traffic, and significant changes in the city. At one time, Springfield was a mining and manufacturing center with heavy demands on freight rail service. Over the years, these industries began to decline. At the same time, state government, insurance, health care and retailing began more and more to dominate the region's economy.

These factors led to a decline in the need for direct rail service to the central business district, and a sharp increase in the number of highway/rail conflicts.

Birth of the Relocation Effort

In 1967, the Illinois General Assembly enacted legislation creating the Capital City Railroad Relocation Authority for the specific purpose of relocating the railroads from central Springfield. The Authority was granted broad powers to achieve this goal—including the right of eminent domain—and to obtain financing for the project.

In 1968, the Authority initiated an extensive study to provide a long-range plan for the relocation of the railroads from central Springfield.

The project received a major impetus in 1972 when the Federal Railroad Administration began a national study of urban railroad relocation. Springfield was one of three cities originally selected as a demonstration city.

In 1973, the Authority recommended a new multi-track railroad corridor located to the South and East of central Springfield (Figure 4). The total project cost was estimated to be $48 million.

Studies continued through the 1970s and into the early 1980s. This culminated in 1982 with a final selected corridor that was shifted farther to the East, East of Interstate 55. The estimated cost of the project was approximately $200 million (Figure 5).

It soon became obvious that there was not enough money to fund the entire project. The FHWA then suggested dividing the project into "usable segments," with the idea of targeting the most critical segments for completion.

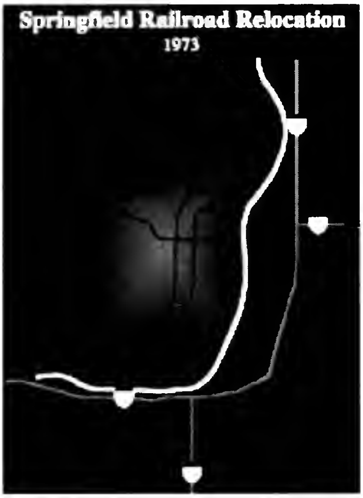


Figure 4. Springfield Railroad Relocation, 1973.

Figure 5. Springfield Railroad Relocation, 1982.

Figure 6. Springfield Railway System, 1989.

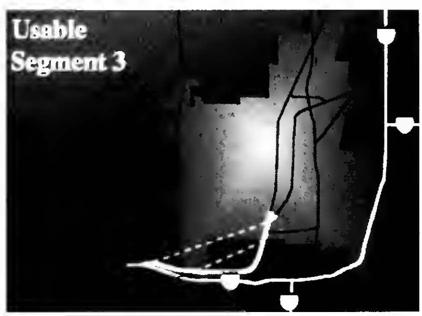


Figure 7. Usable Segment 3.

Usable Segment 1 relocated the Norfolk & Western/Illinois Central Gulf interchange track and permitted the construction of a portion of Madison Street, a major thoroughfare in downtown Springfield.

Usable Segment 2 provided a railroad grade separation structure to carry the connection of Madison Street to Clear Lake Avenue over the Illinois Central Gulf tracks, eliminating a grade crossing.

While rail consolidation and abandonment had significantly reduced the number of tracks in Springfield (Figure 6), two lines on the Southwest side of the city, the Norfolk & Western (now the Norfolk Southern) and the Illinois Central Gulf's (now the Southern Pacific) Kansas City main, had always been a significant barrier to development. These lines also created some of the most dangerous at-grade crossings and a very narrow underpass at a major North-South street, Chatham Road.

Effort Focuses on Usable Segment 3

In an effort to correct this problem, the Authority decided to study the possibility of constructing a segment of the project that would allow relocation of these two lines. This project became Usable Segment 3 (Figure 7). A summary report on Usable Segment 3 was completed in 1985, and the FHWA approved the project report and environmental assessment in 1986.

The proposed relocation alignment began on the West end as a single-track facility for the Norfolk & Western (Figure 8). It would proceed to the East and would roughly parallel I-72, then cross under Veterans Parkway. The Kansas City main would be realigned to become parallel to the Norfolk & Western. The two main tracks would continue to the East, and a 6,500-foot siding track would be included for the Norfolk & Western.

The alignment then turned to the North and followed the existing Illinois Central Gulf St. Louis main. Two 40-car switching tracks and an industrial track were included. The two mains would continue North to the old Iles crossing near Iles Avenue.

This crossing would be replaced with a pair of #20 crossovers. Mainline tracks would be designed to allow 60 mph for freight trains and 79 mph for passenger trains using the St. Louis main, which is the Amtrak line. New grade separation structures were required at Veteran's Parkway and Chatham Road. The project would eliminate three railroad grade crossings and greatly improve a fourth, all of which had a history of accidents. Total length of the project would be approximately 5 miles. The construction cost was estimated at $22 million. Funding was to be provided by the federal, state and local governments, no railroad funds were involved.

Funding Issues Nearly Derail Fledgling Project

Hanson Engineers began surveying, design and preparation of land acquisition documents in 1986. A soil survey was undertaken and a track foundation study was made. The rail bed design included computer modeling at the University of Illinois to determine the required roadbed structure, ballast thickness and sub-ballast design.

At this point, however, the project took a few unanticipated turns. In late 1986, the Illinois Central Gulf sold its Kansas City and St. Louis lines to the Chicago, Missouri and Western (CM&W), a newly formed corporation. Also at about this time, the Norfolk and Western merged with the Southern Railroad, creating the Norfolk Southern (NS). While the effect on operations was negligible, the merger meant new people were soon coordinating the project for both of the railroad companies.

This was complicated by the fact that the project also required cooperation from four highway agencies: the Illinois Department of Transportation (IDOT), the City of Springfield, Sangamon County and Woodside Township, all of which had facilities to be reconstructed or relocated. In addi-

Figure 8. Usable Segment 3.

Figure 9. Veteran's Parkway Structure.

tion, revised estimates put the construction cost well above the $22 million originally anticipated. This increased the local share beyond the ability of local governments to fund it. The project was put on hold until funding issues were resolved.

In late 1988, increasing costs forced the Authority and IDOT to take a hard look at the project with the goal of reducing costs. In view of the greatly reduced traffic on the CM&W, it was recommended that the dual main lines originally planned to serve the CM&W and the NS be reduced to a single main line owned and operated by the NS with the CM&W as a tenant. A reduction in the number of interchange tracks in the north-south corridor was also recommended. These changes offered considerable cost savings to the project without any reduction of facilities to the NS. The CM&W would retain its St. Louis main and would be granted trackage rights for its Kansas City traffic.

IDOT agreed to provide the local agencies share of the funding. At that point, IDOT and the FHWA became the sole funding sources for the project. The total cost was estimated to be about $30.8 million.

Relocation Effort Back on Track

By the end of the year, an acceptable track plan had been worked out. In August 1989, the Memorandum of Understanding detailing the roles and responsibilities of all the parties was signed by the Authority; IDOT; the city and county; and the railroads. The project finally had the necessary funding and agreement by all participating parties.

A lot of details remained to be resolved, but we were going to build a railroad.

In September 1989, a contract was awarded for $2.7 million to construct the new Veteran's Parkway structures. These consisted of two new steel I-beam bridges over the new corridor immediately south of the existing crossing (Figure 9). Work proceeded according to schedule and was completed in May of 1991.

Meanwhile, the state of Illinois had made a financial commitment to assist in an upgrade of the CM&W's St. Louis line, the Amtrak route from Chicago to St. Louis. This paved the way for the Southern Pacific to acquire the CM&W's assets in the corridor in November of 1989. In June of 1990, the next construction contract was awarded for $3 million for construction of the new bridge at Chatham Road. This new five-span structure would cross the relocation corridor, a frontage road and I-72. It would replace an existing parallel structure built to cross I-72. The project was completed according to schedule in October of 1991.

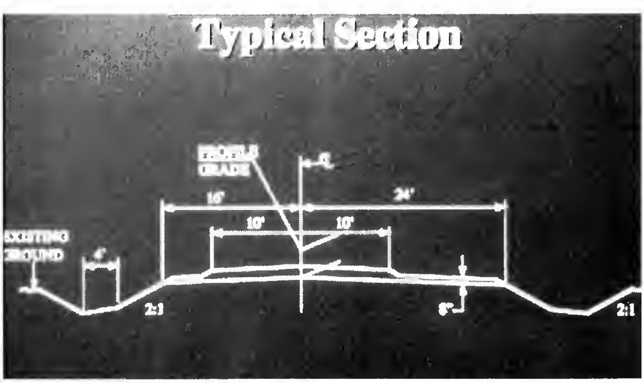


Figure 10. Typical Grading Section.

In order for construction to match funding availability, the project was broken down into a series of contracts. The mainline railroad construction was divided into East and West segments, and the West segment was further divided into grading and trackwork contracts. The West grading contract was the next segment to be designed and constructed.

The typical section for the grading construction included a roadbed 40 feet wide which allowed for a service road on one side (Figure 10). Ditches would be 4 feet wide with 2:1 side slopes, and grading work would include construction of 8 to 12 inches of sub-ballast. Although land acquisition problems delayed this grading contract, it was finally awarded in April 1992 for $1.3 million and was completed in December of 1992. The project included 60,000 cubic yards of earth excavation and placement of 52,000 tons of compacted subballast. Drainage structures were also included in this contract.

The new alignment was graded and compacted. Well graded crushed rock with a 1" top size was placed on the prepared subgrade and distributed with a spreader box. It was rolled and compacted to 100% of standard proctor density, and graded to the required thickness and cross slope (Figure 11). Compaction of the sub-ballast was checked at regular intervals.

Plan preparation then began on the West trackwork plans. The typical section called for 15 inches of ballast below 7 inch by 9 inch ties with 136 lb. welded rail (Figure 12).

In order to simplify the project, reduce required inspections, and take advantage of the railroads volume purchases, it was agreed that the railroad would provide all of the ties and track material, and that the ballast would be provided by the contractor from pre-approved sources.

In early 1993, a contract was awarded for construction of the West trackwork. Total cost of this portion of the project was $4.7 million, including materials.

This contract consisted of constructing ballast, ties and track on the prepared sub-ballast. It included 26,000 feet of track, one #10 turnout and two #20 turnouts. A total of 42,000 tons of ballast was required. Most of the track construction was NS track and the NS's standards were used for its track, while the SP constructed to its own standards. A #20 turnout was installed for the connection of the NS siding, and for the SP's Kansas City connection. Fabric was used beneath the ballast at the turnouts, and underdrains were installed at the road crossings. The contractor constructed the skeleton track directly on the sub-ballast and then dumped the ballast from cars and pulled the track up through the ballast (Figure 13).

Continuous welded rail was used. The NS requested that the rail be bolted by the contractor. NS rail-welding crews would return and weld the joints at a later date, as it turned out about a year and a half after track construction. Other than the spike I bent while trying to swing a hammer, construction went smoothly.

Figure 11. Completed Subballast at Veteran's Parkway.

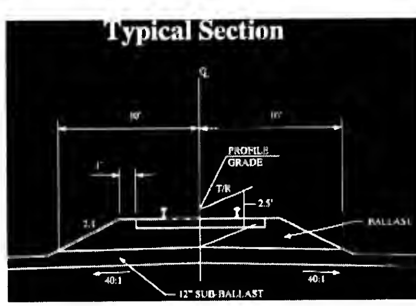


Figure 12. Typical Trackwork Section.

Effort Reaches Important Milestone

The SP's Kansas City traffic was cut over onto the new alignment on December 4, 1993, a red-letter day in the railroad relocation project. After years of discussion, a railroad in Springfield had finally been relocated. This cutover allowed removal of the at-grade crossing of the SP at Chatham Road. A temporary roadway detour was built to maintain traffic.

In July of 1993, a contract was awarded for the construction of the grading and trackwork in the East corridor. This was the largest construction contract, containing most of the trackwork and also the reconstruction of Iles Avenue. The NS would also be installing the signal system with its own forces at the same time. The signal system for the entire joint corridor was to be centrally controlled by the NS out of its office in Decatur. The total value of this portion of the project, including railroad supplied materials was approximately $9.5 million.

The chief challenge of this portion of the project was to reconstruct the two existing SP tracks while maintaining the SP's St. Louis service, and then construct the new tracks adjacent to the existing without disrupting traffic. It was decided that the SP would begin by upgrading its existing East and West mains, replacing rail and ties and raising the profile about 6 inches (Figure 14). At the same time, it would shift the alignment to match the new wider track centers. One of these tracks would become the new NS main and the other the SP main. Turnouts and crossovers would be constructed by the contractor adjacent to the active track and installed by the SP during windows in traffic. Since one of the tracks would be an NS main and the other an SP main, the contractor would construct half of a crossover to NS standards and the other half to SP standards. This upgrade, which required undercutting about 6" of existing ballast, was completed very efficiently by SP crews using on track equipment. After completion of the track upgrade by the SP, the contractor excavated to within 5 feet from the centerline of the active track, placed sub-ballast and replaced the ballast shoulder. The contractor would then excavate the remainder and place the sub-ballast and ballast.

The project included over 50,000 yards of earth excavation, construction of 19,000 feet of track, two #10 turnouts, three #20 turnouts, one #10 crossover and four #20 crossovers. Thirty-six thousand tons of subballast and thirty-three thousand tons of ballast were also placed. This contractor chose to pre-spread 10 inches of ballast prior to constructing skeleton track (Figure 15).

Project Enters Final Phase

The removal of the existing crossing and the cutover and connection at Iles Avenue would be the final railroad phase of the project. Iles Avenue was closed for 60 days for roadway reconstruction and for the final cutover of traffic. Iles Avenue was realigned and completely reconstructed to match the revised railroad geometry (Figure 16). This allowed a significant improvement in safety, sight distance and operating speeds on Iles.

Figure 13. Skeleton Track Construction.

Figure 14. Southern Pacific Tie & Rail Replacement.

Figure 15. New Track Merges with Existing in Joint Use Corridor.

Figure 16. Iles Junction.

Both railroads would be out of operation during cutover, and it was decide to minimize this time as much as possible. Track panels were constructed by the contractor and were ready to be inserted as soon as the old track was cut.

Sunday morning, Oct. 16, 1994, was selected for the cutover. The two railroads and the contractor had a total of over 80 persons at Iles Junction on the big day. They began at 5 a.m. with the contractor removing a section of each track where the new track would connect. He then excavated to the subgrade, compacted it and placed ballast and sub-ballast (Figure 17). At the same time, the railroads worked on pulling the new, previously constructed track panels into place. Panels were set in both the NS and SP track. When this was accomplished, the panels were cut to the exact length and bolted, and ballasted in place (Figure 18). Both lines were surfaced and ready for operation by 3 p.m. that day. All signals were in place and operational and both railroads began operation on the new route. Pavement was completed for Iles Avenue later that month and it too was re-opened to traffic.

All that remained was removal of the underpass on Chatham Road at the old NS alignment. This was completed last June.

In the Final Analysis

Construction has now been completed and the railroads are now operating on the relocated tracks. The SP and NS have salvaged most of their old track.

So what have we learned from all of this? First, some engineering thoughts. Given the weight of the rail used, and the use of #20 turnouts throughout the project, it is obvious that the railroads are looking to construct their facilities for high speed and high tonnage. Efforts were made to minimize long-term maintenance costs in the design of the facility. The SP's Chicago to St. Louis corridor is now being considered for future high-speed development.

We learned that it is simpler to upgrade an existing track than to construct a new one when working near active tracks, even if this means undercutting, complete tie and rail replacement, and realigning the track. The ability to use large, specialized, on-track equipment was critical to this process. The decision by the NS and SP to rehabilitate two existing tracks rather than completely remove them down to the subgrade resulted in significant cost savings, not to mention the savings in time and the reduced impact on operations.

The most important lessons we learned, though, are related to project organization. A project of this type, which requires the cooperation of a number of large public and private organizations, needs a lot of thought at the beginning about coordination and communications. Everyone's roles and responsibilities need to be clearly defined. The parties should sit down together early in the process and start to understand each other's viewpoints regarding the project.

Figure 17. Cutover at Iles Junction.

Figure 18. Placing Track Panels at Iles Junction.

This project was completed in large part due to the dedication and hard work of the people working the closest to it. The railroad and IDOT staff immediately involved with design and construction were able to fight through the problems and setbacks to get it accomplished. Credit also goes to the Capital City Railroad Relocation Authority, which had the vision to see the need for the project and the tenacity to stick with it for over 30 years to bring it to completion.

LONG ISLAND RAIL ROAD AMERICANS WITH DISABILITIES ACT IMPROVEMENTS AND CHALLENGES

By: R. Beck*

Introduction

Pursuant to the mandates of the Federal Americans with Disabilities Act (ADA), the MTA Long Island Rail Road (LIRR) has in recent years completed several initiatives to make its commuter rail system accessible to individuals with disabilities. Bringing a 160-year-old railroad into compliance in a limited time period has required a lot of teamwork, research and innovative solutions. This paper gives an overview of the LIRR's major ADA compliance accomplishments.

Overview of LIRR

The LIRR is the nation's busiest commuter railroad. On a typical weekday, approximately 250,000 customers are transported primarily between suburban Long Island and New York City. The LIRR network consists of 701 miles of track and 134 passenger stations organized along 11 operating branches. Annual estimated ridership is on the order of 72.6 million customers (see Figure 1).

Service is provided by a fleet of 932 electric, multiple unit cars which serve the core, daily commuter territories. Outlying branches are served by 195 conventional diesel hauled coaches.

A unique feature of the LIRR is its use of three major city terminals and a central hub in Jamaica. Trains to and from Penn Station in Manhattan, Flatbush Avenue Terminal in Brooklyn and Hunterspoint Avenue Terminal in Queens make timed connections at the Jamaica hub for trips to and from the outlying branches.

ADA Regulations and the Key Station Plan

The Americans with Disabilities Act of 1990 prohibits discrimination against individuals with physical disabilities. The law is far reaching and affects all types of facilities ranging from public, commercial, governmental and so on. Title II of the ADA pertains to transportation facilities. Both public and employee spaces are covered. This paper will focus on the public improvements which the LIRR has made to its stations. However, reference to related employee functions are made where appropriate.

The first requirement for public transportation entities under ADA was to develop a Key Station Plan. This would designate which stations would become "Key Stations" and be up-graded to comply with all public area requirements of the ADA Accessibility Guidelines (ADAAG). The following criteria set forth by the U.S. Department of Transportation (USDOT), were used to select Key Stations.

a. Ridership—Stations where ridership boardings exceed average passenger boardings on the entire system by 15% were included in this category.

b. Rail to rail connections—This category consists of stations where transfers are normally made between LIRR trains. Such stations include: Jamaica connections; diesel scoot to electric connections; and branch to branch connections.

c. Major interchange points with other modes of transportation—This category consists primarily of bus and subway connections. It includes major points between the LIRR, Long Island Bus, Suffolk Transit, or the New York City Transit Subway System.

d. End Stations—All stations where trains originate and terminate were considered in this category.

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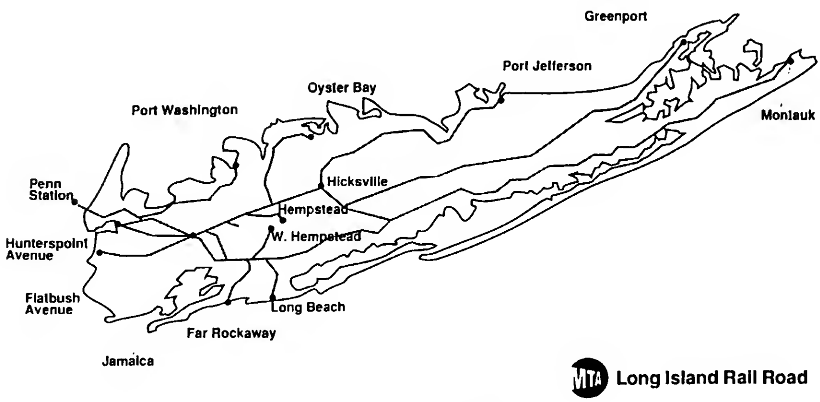

Figure 1. System Map.

e. Major Activities—This is the most subjective category. Stations which are located close to major universities, business or recreational centers. or hospitals were included in this category.

Based on a combination of these criteria, and following a process of public outreach and hearings, the LIRR ultimately designated 18 stations as Key Stations.

These stations are spread out around the LIRR system to make accessible service available to most areas of electric and diesel service. (It should be noted that, over and above the 18 Key Stations, the LIRR has some 40 station platforms which are readily accessible to wheelchair users). More stations are currently being made accessible as part of new construction and restoration projects.

The Key Station Plan was completed by the mandated deadline of July, 1992. With this document in place, work quickly began on completing Key Station improvements by a July, 1993, deadline. A discussion of the major improvements follows.

Accessible Route

An accessible route is required into, out of and through the station so that persons with disabilities can board trains and use various station services (i.e., ticketing, restrooms, train information, etc.). All of the Key Stations have high level (i.e., train floor level) boarding platforms. In most cases, ramps or elevators to access the platform were already existing. At one station, an existing elevator between the station overpass and platform was modified to make an additional stop at street level.

At another station, a brand new pedestrian only grade crossing, complete with gates and lights was constructed to provide an accessible inter-platform connection. At Penn Station, as part of an overall station improvement project, an entirely new entrance, from street level to the LIRR concourse level, was built with stairs, escalators and an ADA compliant elevator. To service the five platforms used by LIRR trains, a new central corridor was constructed with five new elevators running from concourse to platform level. At all Key Stations where restrooms are available to the public, the rooms were reconfigured to provide wheelchair access. In addition to wheelchair access, all obstructions along the accessible route which could be hazardous to the visually impaired were removed or protected by railings. While must station parking lots are either not owned by the LIRR or leased out to other entities, where the LIRR was responsible, accessible spaces were designated and curb cuts installed. The LIRR also coordinated with municipalities or lease holders to make them aware of the ADA parking area requirements.

Signage, Ticketing, Train Information, Phones

ADA also impacts the design of various support and communications services at a rail station.

Signage plays an important and required role in designating the accessible route. Signs displaying the international symbol of accessibility were placed to direct customers to the accessible route (i.e., at a stair pointing toward an elevator).

Information signs of all types were impacted by text size and sign finish requirements. Signs mounted overhead (more than 80 inches above the floor) must use text no smaller than three inches high. This has resulted in an effort to edit messages and display the most crucial information where sign size is constrained by site conditions.

Signs must be matte or other non-glare finish. As the sign finish becomes less glossy and more matte, the signs ability to resist graffiti is diminished. Various materials, including matte over-laminating films and new matte liquid clear coats have been used to create durable signs which are useful to all customers. Care is also taken when placing signs is relation to light fixtures to avoid glare.

Signs with braille and raised lettering are used to designate spaces such as restrooms and the waiting room/ticket selling area. The signs are die-stamped aluminum and have proven to be durable in the outdoor vandalism prone railroad environment. At least one sign with the station name is required on the boarding platform. These have been uniformly located adjacent to the platform ramp or elevator depending on the station design.

Existing ticket window counters were lowered to no more than 36 inches above the floor to accommodate customers in wheelchairs. New ticket windows are required to provide access on the public and employee sides to accommodate a disabled employee. A design is in progress for an adjustable window which provides full height on the employee side when the lower height is not desired.

Ticket vending machines must also be accessible. The bases of existing machines were lowered to accommodate wheelchairs. Braille labels were applied next to control buttons. The instruction panel, containing a code for each of 134 stations could not be accommodated on the machine in braille. As an alternative, a braille brochure is being developed. The next generation of ticket machines will have audio jacks and provide verbal instruction.

If an interior public pay telephone is provided in a transit facility, at least one interior public text telephone shall be provided in the station. The local telephone company, through its licensing agreement with the LIRR and its parent Metropolitan Transportation Authority (MTA), provided a vandal resistant telephone suitable for the transit environment.

The keyboard, for text calls, is located in a steel drawer just beneath the telephone. Dialing another text telephone or a text telephone operator automatically opens the drawer. Hanging up automatically closes the drawer. When not used as a text phone, the unit functions is an ordinary pay telephone.

Where public address systems are provided to convey information to the public, a means of conveying the same or equivalent information to persons with hearing loss or who are deaf must be provided. In order to meet the compliance deadline, a simple system, consisting of one electronic sign above the ticket window was installed. Signs were placed where they could be seen from outside when the station building was locked. The system has a simple menu of messages and is controlled separately from the Railroad's PA system.

Tactile Warning Strips

One of the must controversial requirements of the ADA is the one for placing tactile warning strips along the edge of transit platforms. These strips are specified by the ADAAG as two feet wide with a pattern of "raised truncated domes." Concerns were immediately raised about the potential for slip/trip accidents as well as problems associated with ice build-up and snow removal for outdoor stations in wintery climates.

The LIRR joined forces with the other MTA agencies, NYC Transit and Metro North Railroad to conduct both laboratory and field research. After looking at various materials, the LIRR selected a pre-fabricated, fiberglass panel for its durability and relative ease of installation.

The two-foot-wide by four-feet-long panels were mounted to the platform surface with mechanical fasteners. The perimeter joints were sealed against water infiltration. After allowing time for the sealants to dry, the strips could be walked on within a short time. An important consideration since the platforms could not be shut to traffic during peak hours.

The pre-fab panels also proved very adaptable to various types of platform construction; cast-in-place concrete, precast concrete, asphalt and wood. Water ponding along the edge of the tactile strip was a issue and subsequent to installation, platform drains were drilled. Where drains were not feasible, slits were cut through the tactile strip to form a drainage channel.

Now in their second winter, snow and ice removal has not been as big as issue as anticipated. Manual or mechanical brooms are used to remove snow while de-icers are use to melt any ice that could form. Slip/trip incidents have also not been a major problem.

Rolling Stock

As of July 26, 1995, ADA requires that at least one car per train be made wheelchair accessible. The LIRR electric fleet operates in married pairs and one car per pair has been made wheelchair accessible.

Due to train/platform clearances which must accommodate freight traffic, it was not feasible to meet the horizontal gap dimensions specified by ADA. As an alternative, portable bridge plates are used. These plates are stored on board the train, adjacent to where seats have been removed to provide a wheelchair space. When needed, the plates are manually positioned by a member of the train crew.

The existing diesel fleet has been retrofitted to comply with the one car per train rule. However, the long term plan calls for replacement of the existing diesel fleet with new bi-level coaches. All new rolling stock will be 100% ADA compliant including wheelchair accessible restrooms.

Cost

The LIRR has expended considerable funds to comply with the ADA mandates. Between 1991 and 1999, the LIRR has spent or committed to spend approximately $44.6 million on ADA related improvements. This includes $8.2 million on Key Station modifications, $2.6 million on rolling stock bridge plates and $6 million on ADA improvements at Penn Station. The ADA compliant features of the new diesel coach fleet will cost $9 million with $3.5 million more being spent on making boarding platforms in diesel territory ADA compliant. Many of these projects would have been implemented even without ADA. However, much of the ADA specific expenditures are in the form of Unfunded Federal Mandates for which special Federal funding has not been allocated.

Employee Facilities

While this paper has focused on public facilities, it is important to note that ADA impacts employee areas as well. Of course all new construction must be ADA compliant. However, existing offices must also either be compliant or the agency must accommodate the needs of disabled employees or visitors. This could be done by programming an activity, which a disabled person might attend, for a facility which is accessible. The LIRR has conducted a required inventory of its employee facilities and makes accommodations when necessary.

Conclusion

The LIRR was among the first transit systems in the nation to comply with all requirements of the ADA. Recently, the LIRR participated in the Federal Transit Administration's inspection for ADA compliance and received high marks. This was made possible by the cooperation of all departments at the Railroad working towards this goal. More importantly, the various ADA improvements have resulted in an LIRR system which is more user friendly and comfortable to use for disabled customers and the general public as well.

WHEEL IMPACT LOAD DETECTOR EXPERIENCE ON CN

By: Eric Clegg* and William G. Blevins**

Abstract

CN introduced a system-wide network of 11 Wheel Impact Load Detectors (WILD) sites over the period 1992 to 1995. These detectors are linked to Mechanical Department repair shops to alert maintenance forces to inspect and change out wheels found to produce excessive impacts on rail. Experience has modified original expectations, system configuration and maintenance processes.

What the WILD system achieves is a targeted removal of defective wheels based on the magnitude of impacts recorded. This is the first major step towards truly "performance based" maintenance of wheels.

CN has found that wheel impacts over 100 Kips are highly seasonal, winter month impact rates being about 10 times greater than in summer months. An effective high impact wheel removal program in the summer does not stop high level impacts from reoccurring the following winter. This seasonality is mirrored by dramatically higher winter wheel removals for tread wear and shells, brakeshoe wear, hot boxes, burnt off journals (BOJs), broken wheels and rail breaks.

The WILD system has also found significant numbers of out-of-round wheels caused by tread shelling progressing to the point where tread metal flow smoothes over the shell craters. The result is a wheel which may not be visibly damaged, but produces very high impacts. Some of these can be as high as 199 Kips. CN has adopted a policy of immediately setting out cars found at WILD sites with impacts over 150 Kips.

The 1990's have been a period with a significant increase in wheel damage from shells. The WILD network is a vital operational tool to keep winter rail breakage under control. CN's experience also indicates that BOJs, hot boxes and broken wheels may be controlled by removing high impact wheels. Work is continuing in this area. AAR research also shows long term savings to be made from reduced track structure damage and even less fuel consumption due to reduced drag. Controlling the impact load spectrum through the WILD system also allows informed life decisions to be made for bridge structures.

Wheel Impact Load Detector History

Following a 1984 visit with AMTRAK to look at its application of wheel impact detection technology on their Northeast Corridor, a 1985 expenditure request was initiated by CN Signals & Communications for the purchase and installation of a wheel impact load detector system developed by Salient Systems Inc., of Dublin, Ohio. Field trials were undertaken on the Kingston Subdivision at Lancaster, Ontario to evaluate the electronic integrity of the detector and compatibility to existing CN hot box detector systems. The trials also validated the data output.

After successful field trials, by 1992, additional prototype detectors were placed in service at Westlang (Vancouver), Stony Plain (Edmonton), and Nattress (Winnipeg). The original Lancaster test equipment was later relocated to Vandorf (Toronto). The purpose of additional site installation was to further research and evaluate variable traffic loadings and environmental conditions peculiar to different areas on the CN System. The cumulative data from these prototype sites provided conclusive proof that the WILD system gives valid alarms (i.e. that a defect exists and the axle location is correct). The findings were so conclusive that CN Technical Research (now CANAC-RTF) and Motive Power & Car Equipment (now CN Mechanical) made a joint presentation to the AAR Mechanical

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Management Committee in November 1992. The objective of the presentation was to convince the AAR to accelerate the process to change AAR car Interchange Rule 41A, which defines the criteria for removal of condemnable wheels. Up to that time there had been no provision in this rule to include wheel impact loading as a direct cause for wheel removal.

In 1993 the AAR modified Rule 41A with a minimum condemnable limit of 100 Kips. CN continued its implementation program for five more detectors (plus one relocation) to improve coverage at gateways into major rail terminals on CN. In the same timeframe, an interface was developed by Salient Systems to take advantage of Automatic Equipment Identification (AEI) technology, and this was incorporated into their Mark 2 system. All older version detectors were subsequently upgraded with Mark 2 systems to utilize the positive car identification feature. As well, the new Mark 2 allows peak load readings up to 200 Kips (the older Mark 1 system saturated at 101 Kips). The current detector network was completed in 1995 with the addition of two sites which share impact data with VIA Rail Canada.

Wheel Impact Load Detector Description

The WILD detection system consists of a 200 foot section of tangent track equipped with concrete ties and 132 or 136 pound rail. A set of wooden transition ties with guard rails are installed on each end of the concrete section. The guard rails are used to centre the wheelsets at higher speeds as they come into the measurement zone. The 200 foot section of concrete ties provides a stable base for wheel impact measurements to be taken throughout all climatic conditions. The actual measurement zone, roughly midpoint in the concrete tie section, is approximately 30 feet long, and consists of an array of 10 strain gauged sections mounted on each rail between ties. A photograph of a typical site is included as Figure A. Integral leads from the gauge coupons are terminated into a Wheatstone bridge configuration. These are tied to multi-conductor shielded cable connected to signal conditioning amplifiers or front end processors (FEP's), which in turn feed a central processing unit.

Figure A. WILD Site Photograph.

As a train passes over the gauges, the axles are counted, AEI tag details acquired, stresses logged, and data processed. If alarm thresholds are reached, a report (see Figure B) is generated to a user defined location, such as a car shop printer, over standard phone lines. The exception report contains information, such as detector ID, time, train direction, train speed, locomotive and car count, axle count, train tonnage, as well as the defective axle and car number with nominal, dynamic, and peak impact readings.

Rationale for Wheel Impact Detection

New rail and wheel purchases alone represent an annual expenditure of $130 to $140 million. Savings through enhanced management of wheel, rail, and structure loadings represent a significant potential contribution to company operating ratio improvement. Given the known sensitivity of rail to cold temperatures, it is clear that a detection system is productive in controlling the vertical impact environment the track structure must endure.

The benefit originally foreseen in establishing a network of detectors was an anticipated reduction in stress on track, bridge, and roadbed infrastructure. By early detection of difficult to see wheel tread anomalies, serious damage to wheel, bearing, and other car components could also be avoided as well as the potential for derailment. Improvement in car rolling resistance characteristics would be a by-product of defect free wheel surfaces.

It was envisioned that a net cost reduction could be achieved through increasing or targeting priority wheel replacement in order to realize the desired improvements in the rail/roadbed infrastructure, safety, and fuel economy.

Inter-Functional Project

From the outset of the main project thrust in 1992 there has been close co-operation between the major stakeholders of the system. The relationship was started through the formation of a Wheel Impact Detector Committee comprised of members from the Signals & Communications, Track & Roadway, Mechanical, and Transportation functions as well as CANAC-RTF. Track and Signal engineering were primarily responsible for installation and maintenance of the detectors. The Mechanical

WESTBOUND Train passed Dugald, Track (ONE), Detector at 17:36 10 Mar 96
Train speed = 57.7 MPH
3 Locos and 75 Cars counted
Loco axles = 12, Total axles = 312
Gross tons - Loco(s): 420 Car(s): 7382 Total: 7803
Car tags read: Lead loco #1 tag: CN 9516

REPORT 2 -- Wheel Load Exception Report

Axle	Car/W	B-Axle	Type	Nom	Dyn	Ratio	Peak	DL	RL	PL
54 S	11 / 2	L3	Long Fr. Car	38.1	12.0	1.3	50.1			
N		R3	CNIS 413348	38.1	82.0	3.2	120.7			D
67 S	14 / 3	L2	Long Fr. Car	52.5	74.1	2.4	126.6			D
N		R2	CN 414892	52.5	104.8	3.0	157.3			E
91 S	20 / 3	L2	Long Fr. Car	29.4	56.6	2.9	86.0			
N		R2	LVRC 4225	29.4	99.5	4.4	128.9			D

Figure B. Sample WILD System Wheel Load Exception Report.

Department established the WILD reporting and wheel removal criteria as well as effecting wheel inspections and change-out. Transportation ensured that procedures and guidelines were in place to ensure the safe and reliable movement of trains. CANAC-RTF was commissioned to perform specific research such as car defect growth rate tracking and metallurgical analysis on a number of wheelsets having significant defects.

Through the Committee structure, all of the network operating criteria or objectives were defined, debated, approved, and implemented. The results of ongoing research and data analysis were reviewed periodically so that both system and procedural performance objectives could be evaluated and adjusted where necessary. From time to time both supplier and other system users were invited to share experience, and discuss issues of concern. Overall the Committee process has allowed a high level of success in implementing a detection network that has evolved beyond the original system concept.

Original WILD System Concept—1992

With the amendments recommended, and changes to AAR Rule 41A anticipated, a plan was formulated to develop a network of detectors working in standalone mode and reporting to local shop facilities. In effect, WILD would become a maintenance tool to enhance the visual inspection of car and locomotive wheels.

The general concept set the alarm level threshold to 80 Kips. Any CN car with an alarm at this level would be inspected. If shelling or out-of-round dimensions were condemnable, the wheelsets would be removed. A second level criterion of 100 Kips was to be used on non CN owned cars meeting AAR's interchange condemnable criteria.

Reporting was structured such that alarm reports were sent to the next car shop facility in the direction that the train was travelling. Hard copies of exception reports were also transmitted to a central printer in the CN headquarters office. It was anticipated through this process that the worst offending wheels would be removed, and that the population of wheels with defects would progressively decrease.

CN's WILD Network

CN considers itself a leader in the application of wheel impact detection technology with a network of 11 sites located at the approaches to six major rail centres across Canada (see Figure C). Although other roads may have higher traffic volumes over individual systems, CN now has detection coverage on its core traffic routes from coast to coast, and reports all data to a central database. Sites in eastern Canada at Bagot and Cedars protect the Montreal centre and Vandorf, Brampton, and Aldershot protect the Toronto centre. In Western Canada, Dugald and Nattress protect the Winnipeg centre and Uncas and Stony Plain cover the Edmonton centre. Near the Pacific coast, Nechako protects Prince George and Arnold protects Vancouver. In addition to the central data collection facility, impact reports are distributed to 14 car shop printers and 3 rail traffic control centres. Site status is monitored on a 24 hour/7 day basis at the Montreal S&C Network Management Centre.

CN experiences an increase in occurrences of high impacts in the winter months with extremely cold weather conditions and consequent rail failures. The continuous monitoring of alarm reports allows CN to keep a tight handle on incidences that could lead to service disruptions.

CN Experience with Wheel Defects

AAR Rules

Wheels and wheel maintenance represent the largest single component of expense for freight cars. Wheel design specifications and the field standards for removing from service are covered by Association of American Railroad (AAR) Interchange Rules. All railways in North American must abide by these rules in order to freely interchange cars across the continental network. These rules evolve with time taking into account new developments in materials, dynamics research, and processes.

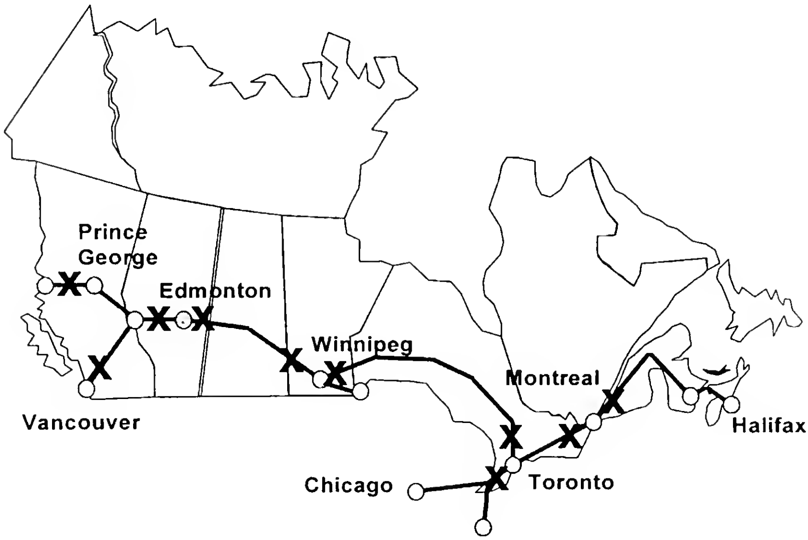

Figure C. WILD Approaches to Major Terminals.

The Wheel Impact Load Detector (WILD) is one of the processes which has been recently adopted by the AAR as a tool for managing the condition of wheels in service. WILD is the first major step towards true "performance based" criteria for a significant expense such as wheels. Traditional causes for wheel removal (AAR Why Made Codes) usually involve the use of dimensional gauges to find wheel defects exceeding specified criteria. An example is a High Flange. AAR Rules require removal if the flange height exceeds 1.5" (AAR Rule 41.A.1.c, WM Code 64). This defect is the result of tread wear and wheels changed out on "foreign" cars may be billed (cross-changed) to the car owner. There are currently 32 different AAR Why Made Codes which govern the removal of wheels from continued service.

Certain defects on wheels are known to have a damaging effect upon track from the repetitive vertical impact forces they impart to the rail and track structure with each revolution. The principal rail impact related defects are Shelled Tread (WMC 75), Out-of-Round (WMC 67) and Slid Flat (WMC 78).

Shelled Tread is a metallurgical failure of the surface (and just under the surface) of the wheel steel material. Two principal causes are:

1. Brittle martensitic formation due to local overheating during a small wheel slide.
2. Fatigue failure of the steel with combinations of vertical loads and lateral/longitudinal tread creep.

AAR rules currently limit the size of shells on the tread to 1" length and width for a single shell, or ¾" if "more or less continuous" around the wheel (AAR Rule 41.A.1.i).

Out-of-Round is limited by AAR standards for wheel manufacture to a radial run-out of 0.030". AAR Interchange Rule 41.A.1.s has, through 1995, limited in-service wheels to 0.050" run-out as measured by an AAR approved gauge. Until the advent of WILD technology, this has not been a cause for field removal of a wheel. It is a defect which is difficult to find visually, and relatively unknown to field forces.

Slid Flat is a well known defect which is governed by AAR Rule 41.A.1.1 under "Handling Line Responsibility." The principal removal dimension is a flat over 2" in length.

Wheels and Impacts

Traditional thinking has always held that impacts are principally from flat wheels. Shells were felt to be minor contributors and out-of-round was a rare manufacturing defect with no practical way of measurement in the field. The WILD system has totally changed previous perceptions. Flats are now found to be a very minor cause for wheel removals due to the essential elimination of the cast iron brakeshoe. Shells have been shown by WILD to produce significant impact forces when their size grows to AAR condemning limits. Shells have also been shown to change character with time. Repetitive impacts will cause tread metal to flow into the shell cavities, eventually "healing" over the old shell crater. The movement of metal will necessarily change the local geometry of the wheel tread surface so that a smooth but out-of-round surface is left. Even a trained eye could miss the defect during a visual inspection.

Figure D shows a wheel tread with a shelled area. This defect, when found, would be cause for removal and billing under AAR rules. The progression of shells to out-of-round (OOR) was not a commonly known phenomenon until the advent of WILD. Figure 1 shows the radial run-out measurements made on a wheelset found with a 178 Kip impact on one wheel in January 1995. The car has a nominal 27.5 Kip maximum wheel load. CN removed the wheel and had the CANAC-RTF research laboratory analyze the radial run-out and metallurgy of the steel.

The most incredible feature of this wheelset (aside from the out-of-round being almost ¼"), is that the mate wheel on the other end of the axle is virtually perfect. The defective wheel had a condemnable shell which when spotted would cause a car inspector to mark for removal, however it was WILD which actually picked off the car en-route. CN's metallurgical analyses did not show this wheel to be any different than normal production wheels. CN has also similarly analyzed over half a dozen high impact wheels with similar results. This is not to say that all shells and out-of-rounds are confined to one wheel of a pair only. Some wheels do show evidence of damage on both sides, probably the result of a small slide precipitating local metallurgical changes and subsequent fatigue and loss of tread material.

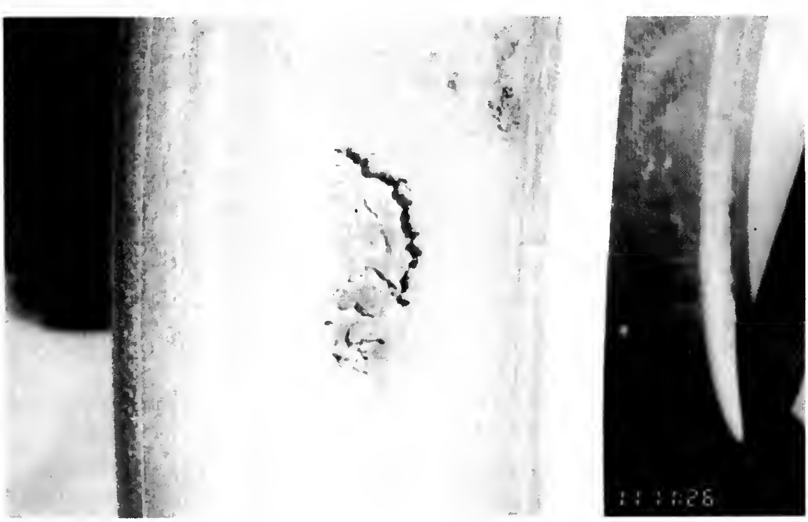

Figure D. Sample Shelled Tread on Wheel.


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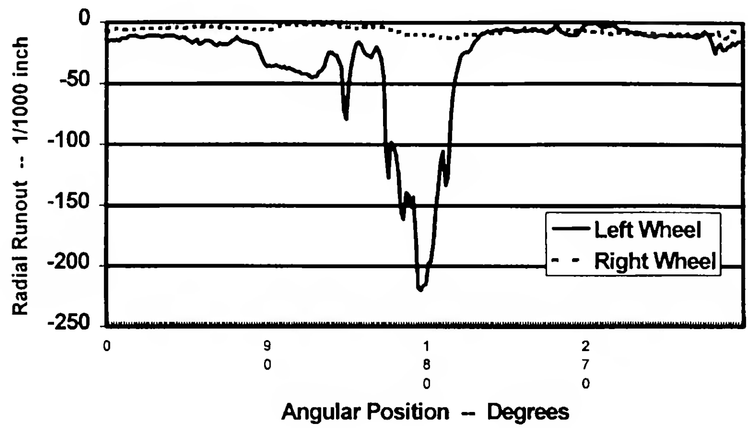

Figure 1. 178 Kip Wheel Out-of-Round CN 606305—Jan. 1995.

Wheel Removal Seasonality

CN has kept extensive records of the reasons for the removal of freight car wheels. A pattern of seasonality is most evident in this data. A disturbing trend is that one type of defect, shelling, has risen from being a minor cause for wheel removal in 1987 to the major cause for removal in 1995.

Figure 2 charts the global history of wheel changeouts on CN's Canadian lines for the 13 year period from 1982 to 1994. Shown are the monthly changeouts for all defects for all cars on line. The changeouts are expressed as Wheelsets per Million Car-Miles. For reference, CN operates about 2,500 million car-miles in a year and changes out about 50,000 pairs of wheels. Referencing wheelset changeouts to car-miles eliminates any bias from normal monthly and annual traffic variations, although these variations are relatively minor in any case.

As is obvious in Figure 2, the winter to summer wheelset consumption variation approaches 2:1 in most years. The seasonality has increased in the 1990's although a slow drop (improvement) is evident in the annual mean consumption rate. The negative implications to workload, repair dwell-time, inventory and production of replacement wheels is clear to any experienced production manager. The "northern tier" railways in North America are all subject to this undesired aspect of winter.

Figure 3 repeats the overall removals but superimposes the cause High Flange, i.e. tread wear. This is the fundamental reason one would expect to change a wheel in an ideal environment. Tread wear is caused by wheel rail creep abrasion and brake shoe abrasion. This cause is also highly seasonal but does not represent the major reason for wheel removal, although by design, it should be. CN's locomotive fleet is equipped with computerized event recorders and extensive download analysis does not show any significant seasonal change in train operation air-brake usage which would explain a 4:1 or 5:1 winter to summer variation in tread wear. The explanation appears to lie with the magnification of the damaging effect of braking in winter compared with summer.

Figure 4 shows Shelled Tread, the cause for removal that has grown to the largest single Why Made Code in the 1990's on CN. Through the early 1980's, shells represented about 10% of removals while in 1995 shells approach 35% of removals. The start of the increase appears to be the winter of 1987/88. A steady increase is evident each winter since then. The same trend is seen in individual CN car series where no known change in weight, service, equipment, reliability, etc. can be found. The

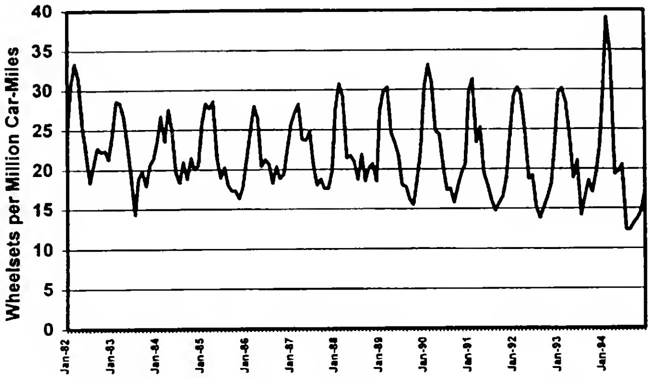

Figure 2. Wheel Usage—Seasonality All Defects—1982 to 1994.

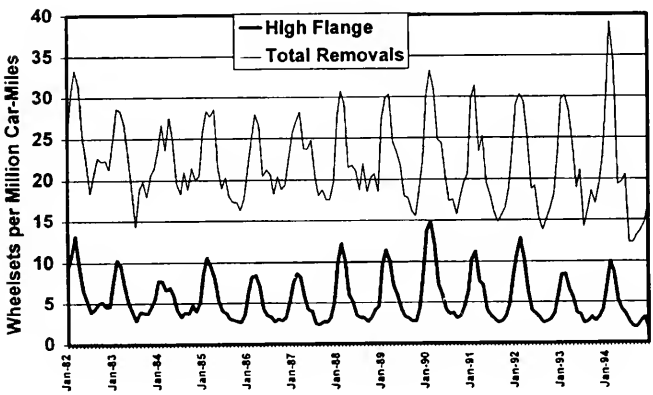

Figure 3. Wheel Usage—Seasonality High Flange—1982 to 1994.

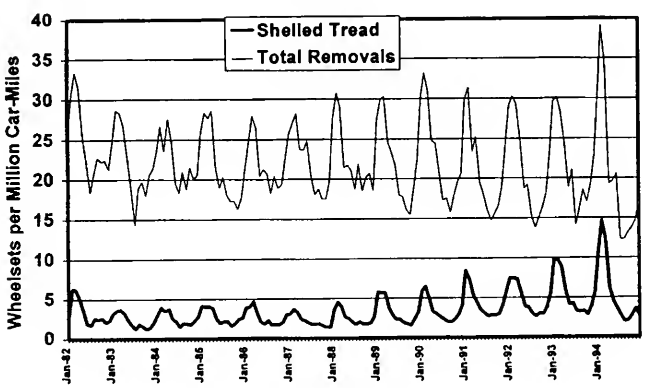

Figure 4. Wheel Usage—Seasonality Shells—1982 to 1994.

causes of this changing phenomenon are not understood at the time. New types of premium trucks, revised wheel metallurgy, changed train handling (driving) habits, etc. may be required to solve the problem whose root cause is not yet explained. CN and the other Canadian railways are actively co-operating together with outside R&D agencies to resolve the issues.

As detailed above, some of the shells will apparently "heal" themselves turning into out-of-rounds, not easily visible. These require remote sensing technology such as WILD to indicate to car maintenance forces that wheels need to be removed from service.

Figure 5 charts the traditional belief of what causes a wheel impact. Skid Flats are a wheel defect cause which is steadily declining due to the removal from service of cars with cast iron (CI) brake shoes. These shoes have a friction characteristic which rises rapidly at slow speed. The "stonewall" stop of CI shoes can produce wheel slide with a heavy brake application or an emergency stop. High Friction Composition (HFC) brakeshoes entered the market in the 1950's and have progressively become the dominant shoe material as older cars are replaced. HFC shoes have a relatively uniform friction-speed characteristic and hence do not normally cause slid flats. Very few flats are now cause for removal but the small remaining quantity is also seasonal, twice as high in winter as in summer.

The seasonality of wheel changeouts is mirrored by the consumption of brake shoes. Figure 6 shows the high winter/summer ratio of about 2:1. The annual average is declining over the years as cars with short life cast iron shoes are progressively retired. The seasonality and the high incidence of "metal pick-up" on the shoe surface in winter is suspected to be related to the high winter wear and shelling rate of wheel treads.

Amidst the bleak picture of high seasonal material consumption, there is one area of beneficial change for both Mechanical and Engineering forces. Wheel removal for Thin Flange is shown in Figure 7 (note scale change for clarity). Thin Flange is an indication of the inability of conventional North American 3 piece trucks to steer adequately in curves. The interesting feature of Figure 7 is that the seasonality is uniquely lower in winter. The incidence of flange gauge face wear grows steadily until mid 1990. Starting mid 1991, a definite downward trend is seen with the mean value of 1994 down to about ⅓ to ¼ of the peak of 1989/90.

The timing of potentially related events such as increased shelling, wayside lubricators, etc., all point to CN's on-board locomotive flange lubricators as being the cause of the thin flange reduction. CN's entire mainline locomotive fleet was retrofitted from 1990–94. These lubricators (KLS) use gyroscopic curve sensors which quintuple the grease application rate in curves. The benefit to rail gauge face wear is obvious but difficult to quantify.

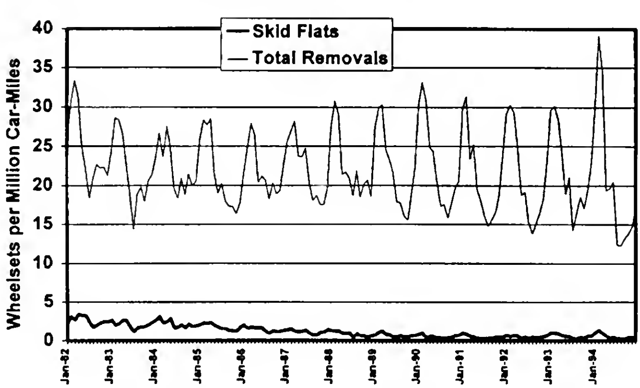

Figure 5. Wheel Usage—Seasonality Skid Flats—1982 to 1994.

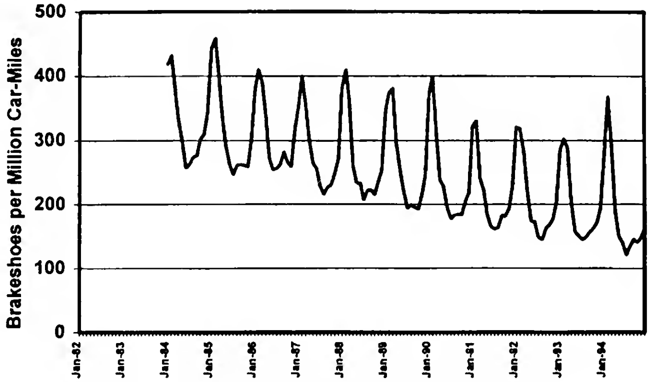

Figure 6. Brakeshoes—Seasonality 1984 to 1994.

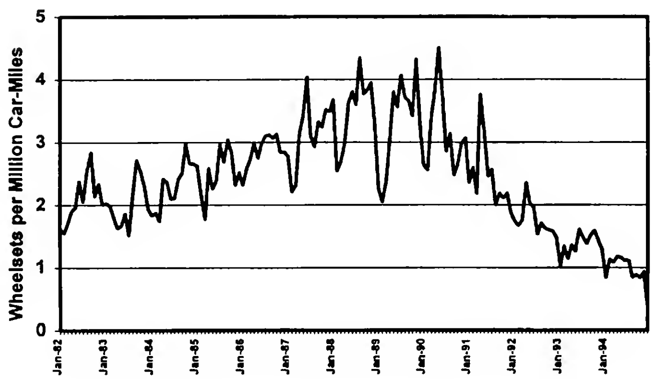

Figure 7. Wheel Usage—Seasonality Thin Flange—1982 to 1994.

WILD Observations—1993 to 1996

CN's WILD network of 11 sites (13 tracks) processes about 4,000,000 wheels per month. As the system has grown, CN has learned that archiving of exceptional wheel impact defects is invaluable in understanding the results. The following sections provide results of monitoring the system since 1993 (4 sites) to 1996 (11 sites).

Distribution of Impact Forces

Figure 8 is a plot of the % of all readings which fall into 10 Kip impact intervals. Ranges are from 0 to 9 Kips, 10 to 19 Kips, . . . , 140 to 149 Kips, and 150 Kips and more. The data is for a one year period February 1994 to January 1995 with 10 sites and 32 million wheels measured. For the observable bars, the wheels fall into 2 main categories: empty cars in the 10 Kip range and loaded cars in the 35 Kip range. Naturally there is a population between for partial loads. Over 87% of readings are less than 39 Kips. Anything above 39 Kips is a wheel with an impact component above its static weight. Above 70 Kips, the small percentage of wheels is not visible on the chart.

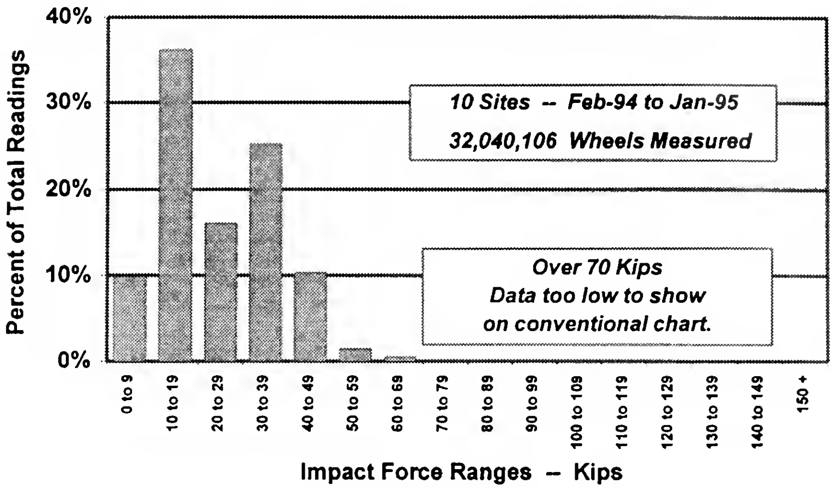

Figure 8. Distribution of WILD Impact Forces % of Total Readings..

A better way to display such data is on a logarithmic scale. Figure 9 is the same data plotted from 0.00001% to 100%. What is noteworthy is that essentially a straight line trend may be plotted for the data above 50 Kips. From this chart, one can determine the spectrum of expected impacts at various levels. The data indicate that the highest levels of impact (150 Kips and above) are very small percentages indeed of the population, i.e. about 0.0007%. One should not equate small percentages with small effect. For 50,000,000 wheels scanned, 0.0007% is 350 per year. Concentrated in 4 winter months, this means that a potential rail breaking wheel is being found at the rate of about 3 or 4 per day. This implies that several are at work continuously doing the opposite of "Non-Destructive Testing" of the plant at any given moment.

Seasonality of Impact Forces

Figure 10 charts the history of wheel impacts exceeding 100 Kips on a monthly basis from January 1993 to January 1996. The plot is the percent of all wheels scanned for the month so that the results are normalized for the growing number of WILD scanners during the period. It is evident that the same seasonal pattern as wheel wear, shelling and brake shoe consumption is at work. On CN the summer impact quantities are 400–500 per month (0.01%). In winter, this rises to 4,000–5,000 per month (0.1%). Each autumn, the population of 100K wheels increases, peaking in the February period and falling off quickly in April. This is a classic demonstration of the defects being generated at a rate which exceeds the ability of maintenance forces to find and remove from service. Wheel removals for shells rise 5 fold in winter but the high impact population rises 10 fold.

What was unexpected in the results over the last 3 years was the degree to which the impacts reoccur each autumn. Over the spring and summer, Mechanical Department forces remove virtually every high impact wheel. The numbers left are so small as to be almost impossible to capture. It had been originally hoped that this successful removal, using WILD to direct field forces to the priority cars, would progressively lower the inventory of wheels for succeeding winters. As is obvious, the shells and out-of-rounds are recreated starting each autumn.

The trend on Figure 10 shows each successive summer and winter being somewhat higher than the previous year. When first installed, CN adopted an alarm and changeout policy for CN cars (non-

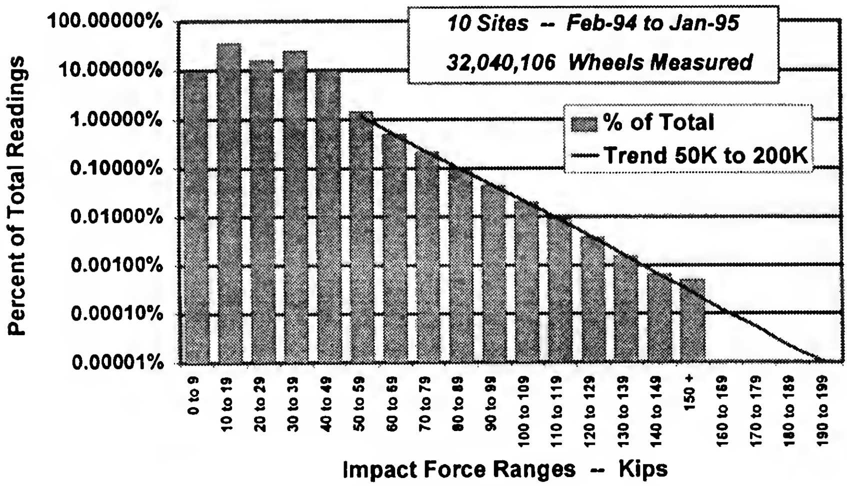

Figure 9. Distribution of WILD Impact Forces Log Display.

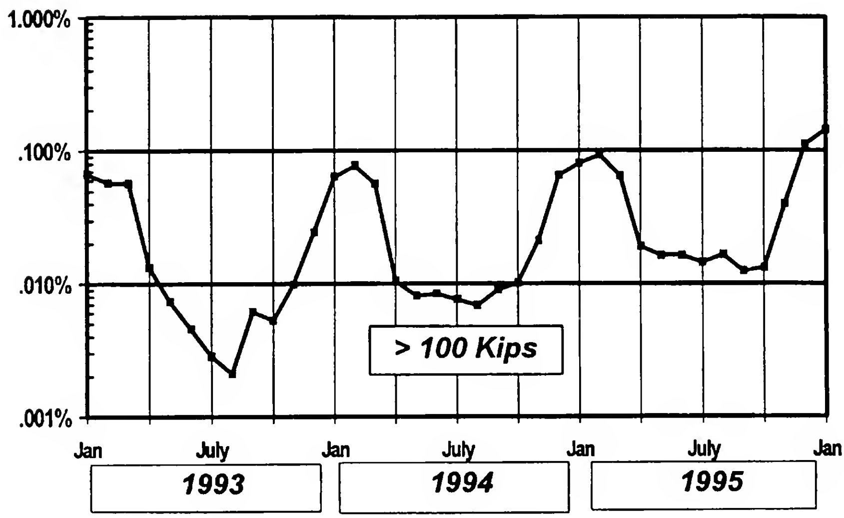

Figure 10. Seasonal Variation Percent Impacts over 100 Kips.

interchange) at 80 Kips, considerably more aggressive than the 100 Kips authorized by AAR interchange rules in the period. The reality of the service and cost implications of this have caused CN to progressively increase the alarm threshold to 100 Kips for all cars. In particular for winter 1995/96, some of the worst weather on record has inflated wheel damage and hence the % over 100 Kip impacts.

The statistical effect of tightening impact levels to 80 Kips is shown in Figure 11. Depending on the season, a 5 to 10 fold increase of alarms and changeouts can be predicted. The economics of such a bold policy would need close examination.

Figure 12 charts the time history of impacts greater than 150 Kips. Summer levels are about 0.0001% or 3 to 4 per month. This rises by Dec/Jan/Feb/Mar to about 0.003% or 120–150 per month. The magnitude of these very high level impacts (5 times static weight) caused CN to take special action with such wheels as will be described below.

Seasonality of Potentially Related Car Defects

Aside from the effect of high impacts upon the track structure, the repetitive nature of impacts (500–600 times per mile) can hardly be beneficial to car components. CN has indications that hot bearings, broken wheels and dragging equipment such as dropped brakebeams have some relation-

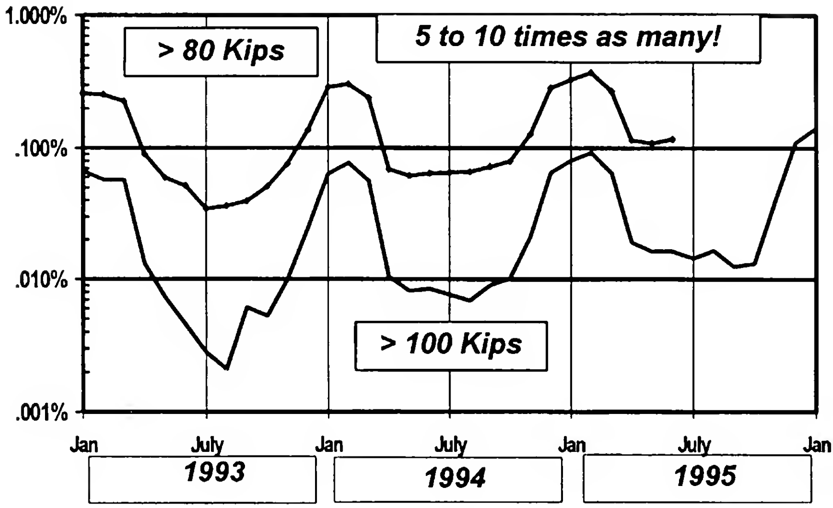

Figure 11. Seasonal Variation Percent Impacts over 80 Kips.

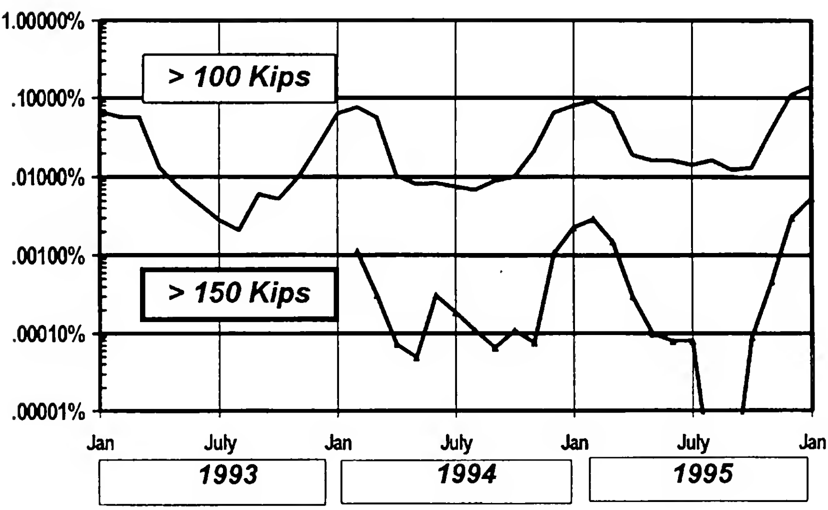

Figure 12. Seasonal Variation Percent Impacts over 150 Kips.

ship to high impacts. Not all such occurrences can be associated, but a significant number can be. The consequence to destroying miles of track from a derailment of a burned-off journal (BOJ), broken wheel, or a dropped brakebeam is of interest to the Engineering function of each railway.

Hot Boxes and BOJs

Figure 13 charts the seasonality of BOJs and confirmed Hot Boxes on a quarterly basis over the period 1986 to 1995. The effect on bearings is clearly seasonal and matches wheel and impact sensitivity to the winter. Currently, CN's network of 410 Hot Box Detectors (HBD) is independent of the 11 site WILD network. The natural variation of the plotted HB and BOJ data makes it difficult to see if WILD should be associated. However, an analysis of specific events makes the association clearer.

Over the 7 month period December 1994 to June 1995, CN experienced 17 BOJs (out of over 800,000 bearings on-line at any instant). Over the same period CN found 0.06% of all wheels scanned had impacts over 100 Kips. Both these events are very small percentages of the opportunities for failure. The question is: what is the chance that both these unlikely occurrences would appear on the same wheel? If one hypothesizes that BOJs and 100K impacts were *unrelated*, then it follows that the 17 BOJs should have the same chance as any other wheel for a 100 Kip impact, i.e. 0.06%. Therefore 17 x 0.06% = 0.01 BOJs should have a 100 Kip impact also. This implies 1 chance every 58 years. In fact, CN found that 5 BOJs had 100 Kip impacts within days of going BOJ. The statistical chance of this if they were unrelated events is virtually nil. One can then logically argue that BOJs and 100 Kip impacts must therefore be *related* in order to have over 30% having both defects. CN is now investigating effective ways of tying together the HBD and WILD data networks to capitalize on the connection found.

Broken Wheels

A declining but important car related derailment issue is broken wheels. Generally these are the product of a stress reversal in the wheel rim and plate. Industry research work shows this is caused by a thermal (braking) event (or events) during their life. Some obsolete wheel designs (straight plate) are far more prone to failure than current production. AAR had mandated accelerated removals

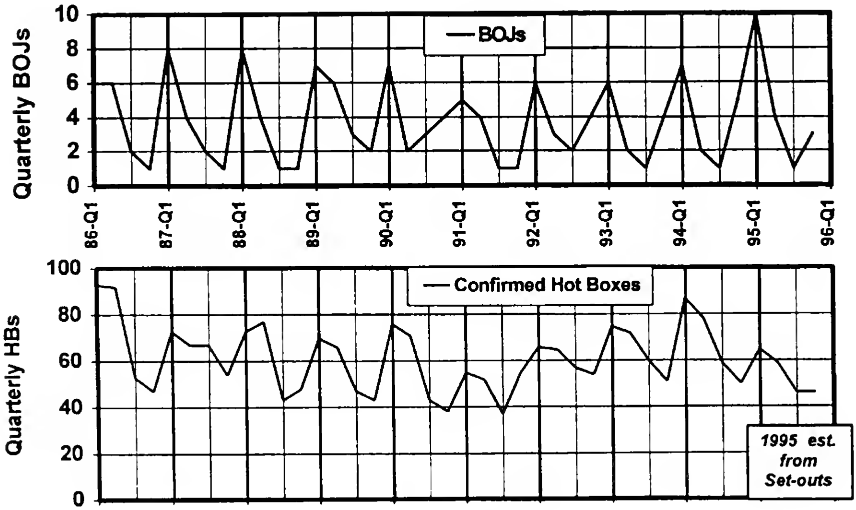

Figure 13. Quarterly Hot Boxes and BOJs.

of straight plate wheels. As the population gradually declines, the risk decreases. However, no wheel is immune from thermal abuse to a level which reverses built in compressive stresses to tensile stresses. With tensile residual stress, a surface defect, combined with repetitive load cycles may result in sudden fracture.

Figure 14 is a plot of CN experience with broken wheels, on a quarterly basis from 1989 to 1996. The tall bars (15 to 20) at the 1st quarter of each year are the broken wheels, found principally on inspection at terminals. A small quantity result in derailment on-line, often 3 to 5 in the 1st quarter. Typically in the 1st quarter of each year 2 derailments can be classed as major (defined here as 3 cars or more). CN has seen a dramatic drop in failed wheels in the first quarters of 1995 and 1996, about ⅓ the level of previous years. An accompanying drop in derailment is also seen. A major derailment has not occurred since August 1994, an unprecedented period of 18 months (at time of writing). Victory cannot yet be declared; however, one could fairly credit WILD for the removal of wheels with a higher than normal chance of wheel failure. Figure 15 is an annual plot of broken wheels and derailment. This again shows the sharp decline in the past year.

Broken Rails

The original concept of WILD was to prevent track and structure damage. AAR Research & Test has published numerous reports on the subject, which document the economic effect of impacts on rail, track components and roadbed. The most difficult aspect of the damage to quantify is broken rail. CN's broken rail experience in winter, like all the other subjects of this paper, exhibits dramatic increase with cold weather. The internal stresses from thermal contraction are well known and documented. More difficult to predict is the association with internal stress raisers, aggravated by fatigue and occasional impacts at just the right point to propagate to failure.

In the last year (1995), CN has put an emphasis on the collection of sufficiently detailed causes of rail breakage. Relationship to ambient temperature, rate of change of ambient temperature, service history of the rail steel and its manufacturer, and impacts, etc. all play a role in current on-going

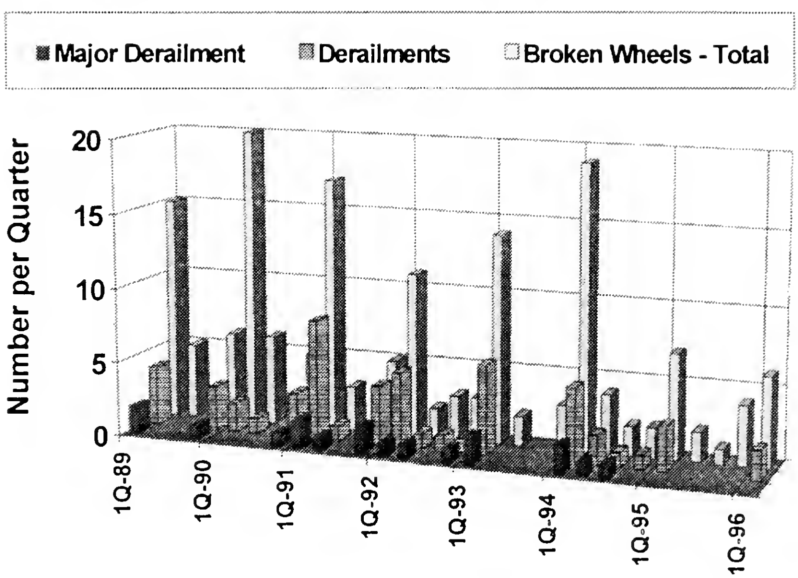

Figure 14. Quarterly Broken Wheels.

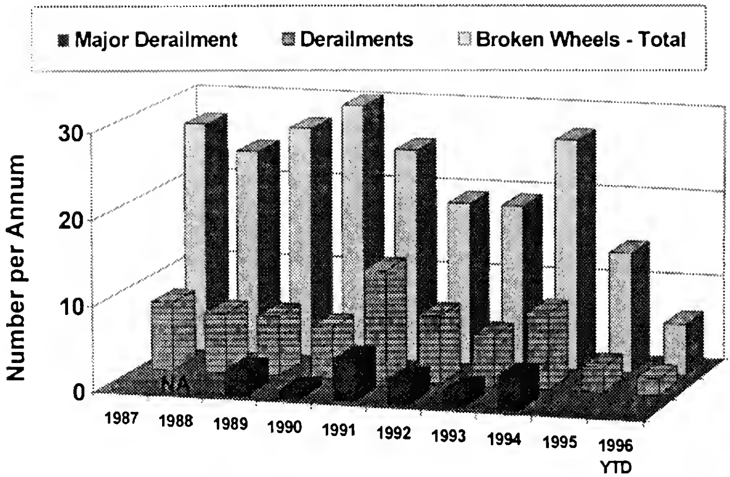

Figure 15. Broken Wheels—1987 to 1996

analyses. Figure 16 is a concurrent plot of impact incidents per 1,000 axles at various impact levels. Overlaid is a CN System plot of monthly rail failures since March 1995. The seasonal pattern is evident. The rail failure magnitude lies between the 125 Kip and 150 Kip impact plots. CN has evidence that ties specific high impact wheels with consequent broken rails en route. Notwithstanding the extraordinary cold winter weather of 1995/96, CN has seen the WILD system control rail breaks to a manageable level. Given the rate of increase of shelled wheels in recent years, without WILD, CN's Engineering forces would have faced substantially more broken rails than we have had. The effects upon service would be devastating if uncontrolled.

1994/95 CN WILD Research Work

As the WILD network grew in 1993/94, CN realized there was a substantial body of data and information available that had not been initially contemplated. Aside from the raw statistics of monthly hits, AEI tags were being added to cars which allowed specific car numbers and wheels to be flagged. No more guess work from an axle number and a cut-list in the yard! Further, the data from the network are now assembled in a central database and real-time data from sites can be ported to destinations other than the local car shop.

This in mind, CN decided to determine: 1) if a 100 Kip impact was a rapidly changing event and 2) if criteria could be developed to pick off an empty car with a potential high impact before the car was reloaded. CN's aim is to prevent delays to customer loads and hence to prevent WILD from becoming a service reliability problem. CN contracted with CANAC-RTF to undertake this work. The WILD system was specially configured to allow CANAC Research to track impact data as low as 30 Kips so that long term growth could be analyzed. CN selected a utility coal service with 300 rotary coal gondolas running from Alberta to Lake Superior (Thunder Bay). The cars have unique reporting marks, travel 2,800 miles in a round trip and pass over 3 WILD detectors each way, loaded and empty. These cars were monitored over a 12-month period.

Impact Growth Research Results

Over a period of 6 months or 50,000 miles, most loaded car impacts, if initially at the 60 Kip level, would not exceed 90 Kips. The corresponding empty car values of 35 Kips would not exceed 65 Kips over the same period. A considerable scatter in data was evident which set in motion an

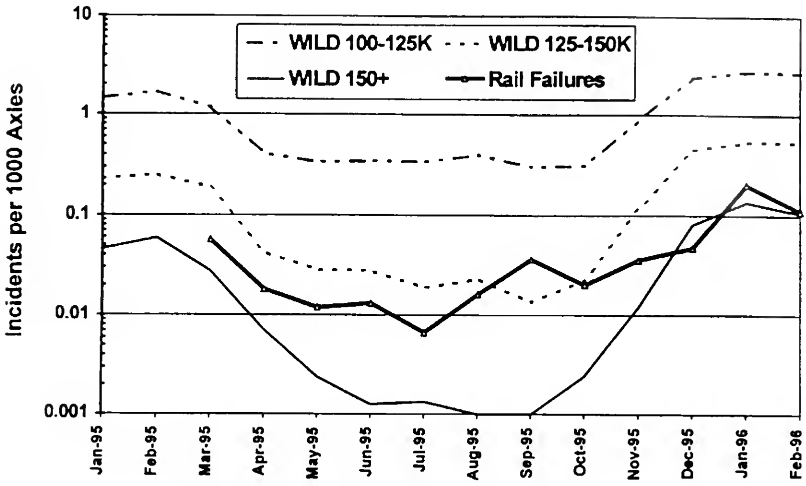

Figure 16. In-Service Rail Failures vs WILD

investigation into repeatability (covered later in this paper). Paradoxically, winter did not seem to drastically increase the measured impact Kip levels. Wheels above the 100 Kip level seemed to increase faster, but still take considerably longer to rise appreciably than was expected. The net result of this investigation is that the movement to destination of a 100 Kip alarmed car will not normally result in a substantial increase in loading before the end of the trip. This has allowed some productive process changes.

Empty Car Criteria Research Results

The WILD system scans each wheel many times as it traverses the multiple strain gauges per rail at each site. Many vertical load measurements are made on an unaffected part of the wheel, so a statistical methodology can make an estimate of the Nominal (static) weight on the wheel. This is done by "tossing out" the high impact affected results. The difference between the Peak Load and the Nominal Load is called the Dynamic Load. The Dynamic Load is fundamentally a value related to the geometric deviation from round and the unsprung mass of the wheelset. Theoretically the Dynamic Load is not strongly dependent upon the sprung weight of the car. In practice there is a small reduction of the Dynamic Load for an empty car compared to a loaded car.

The ratio of the Peak Load to the Nominal is called the Dynamic Ratio. A loaded 263,000 lb. car has a Nominal wheel load of 32,750 lbs. (or 33K). If a 100 Kip Peak impact load is measured, the wheel has a Dynamic load of 67 Kips (100K–33K) and a Dynamic Ratio of 3.0 (100K/33K). When empty, the same car theoretically has a nominal wheel load of 8 Kips, a Dynamic load of about 60 Kips, for a peak of 68K, which is below alarm threshold levels. The Dynamic Ratio is 68K/8K or 8.5. Given that there is a large gap between the Dynamic Ratios for empty and loaded cars, there is a theoretical opportunity to use this to halt empty cars before they become loaded en-route alarms at 100 Kips.

In practice, however, our research shows a problem yet to be overcome. Some wheels near the 100 Kip level when loaded, have damage over an extended part of the wheel tread. When empty, such cars, with true static wheel loads in the range of 7 to 9 Kips, will show inaccurate nominal loads from WILD in the 11 to 14 Kip range. Ordinarily this small inaccuracy would be inconsequential. However, if an inaccurate empty nominal of say, 14K, is divided in to a true measured peak of 68K (example above) the Dynamic Ratio is below 5.0. If this low value became the empty car criterion for wheel removal, many empty cars with small non-condemnable defects and correctly measured nominal weight will become subject to premature inspection. CN is looking into ways to overcome this effect. In the meantime, alternatives to achieve the same end have been developed which will be detailed later in *Process Enhancements*.

WILD Measurement/Alarm Repeatability

In conjunction with the network expansion, training sessions were conducted in the field. These provided an overview of detector operation, as well as detailed instructions on wheel defect identification and removal procedures. During peak seasonal swings, exceptionally high numbers of defective wheels were being identified. Due to the high profile of the removal program, and the large volume of wheel defects, concerns were often voiced from field car shop personnel over the integrity of the data being received. The anxiety expressed was the seeming lack of site to site consistency in impact data. Car equipment staff were responsible for the monitoring of reports for cars coming into and departing a major centre. For example, car CN 414054 would pass detector A inbound with an alarm level of 102 Kips, while on an outbound move at detector B the alarm level would reak 118 Kips. There were numerous instances of this happening across the system, and the immediate conclusion was that the detectors were not correctly calibrated.

An examination of site configuration and calibration was undertaken for all sites, and in fact one site was out of specification and corrected. As a result of the site integrity check, and support from the vendor with an explanation of the dynamics of site data acquisition, it was determined that there are five major influences on impact repeatability.

Speed

One of the manufacturer's criteria in obtaining a good measurement of impact forces is that the site be located in an area where average train speed reaches at least 30 MPH. Some of CN's sites are located near the end of sidings where trains may be accelerating or decelerating over the measurement zone. Vertical force levels generally decrease with low speeds.

Direction

Each wheel tread defect carries with it a geometric signature which may vary the impact magnitude depending on direction of rotation of the wheel. On CN this was evident at one particular rail centre where trains enter the yard on a wye. For example, a train consist moving westward and passing over a detector before arriving for inspection at the yard, would be in reverse order when departing the yard continuing westward over another detector.

Wheel diameter coverage pattern

CN currently has two different WILD gauge layout patterns. A "5-1-5" pattern used at most sites, and an earlier "4-3-2-3-4" pattern at two sites. The 5-1-5 layout means, for example, 5 active cribs at 24" spacing, 1 blank crib, and 5 more active cribs at 26" spacing. Both patterns utilize 10 active cribs and 20 processor channels. Passing over the sites are wheels with diameters ranging from 28" to 42". Each strain gauge layout, or pattern, attempts to optimize wheel tread coverage and not miss a defect as it passes over the measurement zone.

Good coverage distance for a strain gauged crib is based on the output being at least 80% of its maximum. The general principle is to obtain a practical balance between coverage of all major wheel diameters without having to extend the number of channels required at a site to obtain 100% coverage on all wheel sizes. CN has need for multiple sites to cover its transcontinental traffic. If a wheel defect does not always hit on the "sweet spot" of any of the gauges at one detector location, it will in all likelihood be picked up at an adjacent detector site.

Defect position on tread

Observation of removed shelled wheels and other surface defects indicates that the severity can vary across the tread surface. The actual wheel to rail contact surface area is only about the size of a dime and the total lateral wheel displacement can vary from 0.6" to over 1". The trajectory of a defect may not bring it within 100% of the contact surface at the active strain gauge crib, and as a result, the force of the impact may be attenuated.

Car motion dynamics

The motion of a 131 ton car travelling in a freight train at 50 MPH can generate dynamic vertical and lateral force on the rail structure, especially if there is any induced oscillation because of track geometry or car dynamics. Although the layout and design of the detector site has been optimized to reduce or dampen such forces, they can still translate into measurable vertical loads which are read by the detector. Car motion itself is not a predominant factor in the accuracy of the system.

The conclusion reached by CN is that due to the contributing influence factors above, that readings for any given car will vary from site to site, but that a network of detectors will increase the probability of defect identification.

Revised 1996 WILD Network—Based on Service Experience

Based upon the results of CN's practical experience of the last 3 years with WILD, a number of hardware, software and process changes have been underway. The operating experience includes repetitive seasonality, slow growth of impacts in the 100K range, the reality of repeatability variations, the frequency of 150 Kip impacts, and improvements in service delivery to customers through bypassing traditional hump yards and WILD connected car shops.

Hardware and Software Enhancements

AEI

Since WILD was originally installed, AEI has now become universal and allows for definitive pinpointing of the affected wheel. Without a doubt, the addition of AEI and positive car identification, has been the most important improvement in gaining field acceptance and confidence in the detection system.

Network Monitoring and Diagnostics

As built, the prototype standalone systems communicated over standard telecommunication lines. Automated WILD site status monitoring was virtually non-existent. The only means of confirming site health was to manually call up each location and login.

In an effort to reduce communication costs and at the same time provide easier access to a host database, starting in 1993, all sites were converted to X.25 protocol. All report transmission was moved to the CN private network, and copies of exception data were logged on a host UNIX computer. The next step was to develop a means of automating the connectivity status of the detector. This was achieved in conjunction with Salient, who developed a feature which allowed the detector to broadcast a site OK message at a user definable interval. At the CN office, a program was written to monitor the communication lines for data activity from every detector. The monitor now displays site activity in the form of a time bar. If a given detector transmission interval exceeds one hour, an alarm indicator is activated on the monitor and corrective action taken.

There are a number of system internal diagnostic features which can be used in determining the performance of the detection system, most of which require human interpretation and manual intervention. The integrity of the system relies mostly on accurate strain gauge and front end processor operation. It is in this area that the manufacturer is developing a self diagnostic and reporting enhancement which should assist in optimizing system performance.

Centralized Database

The acquisition, storage and processing of impact data for a train move requires large system memory capacity. Because of memory limitation, each site normally retains train data only for the last movement over the detector. A new train entry basically overwrites the previous train record. Because exception report data had such a short lifespan, the decision was taken to log each train record into a central database. The usage of this data has increased exponentially over the period for which train reports have been recorded. What originally started out as a back-up for incomplete transmissions or occasional inquiry pertaining to a derailment or burnt off journal, has turned out to be a powerful car management utility. The data is now put to use regularly in the compilation of impact statistics, car record inquiries, as well as the "2 Strike" record which is detailed in the process enhancements.

Process Enhancements

The frequency of 150 Kip impacts and their connection to increased risk of rail failure has impelled CN to institute a policy of immediately stopping cars with 150 Kip or greater impacts on-line. This feature was not anticipated when the WILD system was instituted. CN's Hot Box Detector dispatching office operators are directly alerted by dedicated printer when 150 Kip impacts occur. The HBD operators request the train to stop and set-out the car at the next available siding. The principle adopted is similar to setting-out a hot box.

CN's research that the growth rate of 100 Kip impacts was not high gave an opportunity to change the traditional approach to car and wheel maintenance. Originally a car would be flagged for investigation at the local maintenance point adjacent to the detector site. With the centralization of the database, centralized planning and control of the disposition of the car can be made. CN refers to this enhanced use of the remote sensing capability of WILD as "Two Strikes and you're OUT!".

When a 100 Kip or greater impact is automatically logged in the central database, an automatic computer search for the car number and wheel number is made for a match within a 30 day window. If two 100K+ strikes are found, an instant message is sent to a 24 hour coverage Mechanical Dept. office which arranges disposition for inspection and repair. Criteria currently being tested include allowable distance to travel which is dependent upon the magnitude of the impacts. The higher the impacts, the sooner the car is stopped for verification per AAR standards. Figure 17 gives a sample of the automatically computer generated "2 Strike" CN E-Mail message. The advantages of the 2 Strike process include:

1. Planning for car inspection and repair at the most logical location.
2. Unnecessary delays to customer loads are avoided.
3. Planning workload and safe movement of loaded cars eliminates the need for an empty car criterion.
4. Repeatability is no longer a debate issue when two 100K+ hits have been logged.

CN's statistics show that the extreme 1995/96 winter months produce about 50 2 Strike automatic messages per day. Summer 1995 produced 3 to 5 per day. The 2 Strike methodology does not yet replace the 100 Kip messaging directly sent to local car shops for single strikes. The two systems currently work in parallel, each contributing to navigating a transition to true performance standards, in a changing environment which demands both customer service reliability and safety.

Future WILD Plans

Database consolidation

CN has a substantial investment in WILD technology and the current WILD system serves the Railway well in detecting cars with wheels having AAR condemnable defects. A prototype ("2 Strikes") alarm system is now in operation, which allows CN to detect wheels which have had multiple alarm readings. This prototype is currently running on a general purpose Signals and Communi-

Date: Thursday, 15 February 1996 10:53pm
From: nls <nls@scoff.cn.ca>
To: blevins@taosmtp.cn.ca, wonnek@taosmtp.cn.ca, posyniak@taosmtp.cn.ca,
gussow@taosmtp.cn.ca, cariati@taosmtp.cn.ca
Date: Thu, 15 Feb 96 22:54:08 EST
Subject: 2_Strikes_Alarm_Car_with_history
Reply-To: nls@scoff.cn.ca

Thu Feb 15 22:53:52 EST 1996 "2 Strikes" Alarm Car with history

Site	Date	Time	R/W	Car No	B	Speed	Nomi	Dynam	Peak	Ratio
W Bagot	96/01/26	18:24	CN	414860	R3	55.1	38.3	72.0	110.3	2.9
W Dugald	96/02/15	22:06	CN	414860	R3	59.5	26.2	82.2	108.4	4.1

-----------(end of letter)---

Figure 17. Sample "2-Strike" Message.

cations office computer. In order to properly manage the wheel fleet, as well as assess the effects of impact forces on the track and bridge plant, the database will be centralized onto a new dedicated host computer system.

Anticipated benefits and additional changes:

1. A database of wheels which have exceeded exception thresholds will be available for historical analysis, allowing repeat offenders and car series which are susceptible to defects to be reported and corrected.
2. Dedicated circuits will be installed between each detector and the host computer. The detector site will be configured to transmit wheel impact data in compressed form rather than a pre-formatted ASCII report. In essence, raw train data will be processed and distributed from a central location, facilitating custom report capability. Alarm messages will be delivered through the CN E-Mail network to printers instead of direct dial-up from the detector. This will allow flexibility in controlling message distribution.
3. Tracking site performance and calibration will improve CN's ability to deliver the most accurate information possible. Individual conditions such as site calibration and running speeds can be analyzed and exceptions reported and corrected.
4. Statistical generation on wheel loadings at various speed ranges over time will allow CN to track the impact performance of the wheel fleet detecting any trends in performance. In addition the database will facilitate analysis of impact related in-service rail breaks.
5. Statistics will be generated from axle weight data at track speed to evaluate stress on bridges without affecting train performance. Existing overload car detectors can be retired.
6. The new host computer will provide a platform on which Hot Box, Hot Wheel, and Dragging Equipment detection data can be integrated.

AAR Interchange Rule Changes

Current AAR rules still focus on traditional "tape measure" criteria to determine whether wheels should continue in service. For those defects which are impact related, WILD offers a performance-based alternative. AAR staff reports indicate that optimized WILD criteria has the potential of a large RR industry net saving. This potential needs a substantial part of the North American network to have WILD coverage to become practical. CN intends to continue working with interested parties to further this aim.

CN Shelling Research/Preventive Work

Wheel shelling prevention is a primary focus on CN Mechanical Department activities. CN has a large number of parallel action plans underway to get at the root causes. These include:

- Wheel metallurgy
- Train air brake handling
- Wheel/Rail profile integration
- Handbrake release enforcement
- Improved truck designs
- Improved brake shoes
- Car repeater investigation
- Problematic car series investigation

These activities are being done in association with other Canadian railways (CP Rail and Quebec Cartier Mining) and research agencies (National Research Council of Canada, AAR R&T, CANAC-RTF). Shared data and progress will assist each railway to home in on causes which can be economically addressed.

Summary

CN's positive experience with Wheel Impact Load Detectors can be summarized in the following points:

1. CN's system-wide network of 11 Wheel Impact Load Detector sites was introduced from 1992 to 1995. These are linked to Mechanical Department repair shops to alert maintenance forces.
2. This is the first major step toward truly "performance based" maintenance of wheels.
3. Experience has modified original expectations, system configuration and maintenance processes.
4. CN has found that wheel impacts over 100 Kips are highly seasonal, winter month impact rates being about 10 times greater than in summer months.
5. An effective high impact wheel removal program in the summer does not stop high level impacts from reoccurring the following winter.
6. This seasonality is mirrored by dramatically higher winter wheel removals for tread wear and shells, brake shoe wear, hot boxes, burnt off journals (BOJs), broken wheels and rail breaks.
7. The WILD system has also found significant numbers of out-of-round wheels caused by tread shelling progressing to the point where tread metal flow smoothes over the shell craters. The result is a wheel which may not be visibly damaged, but produces very high impacts, up to 199 Kips.
8. CN policy is to immediately set out cars found at WILD sites with impacts over 150 Kips.
9. The 1990's have been a period with a significant increase in wheel damage from shells, making the WILD network a vital operational tool to keep winter rail breakage under control.
10. CN's experience also indicates that BOJs, hot boxes and broken wheels may be controlled by removing high impact wheels. Development work continues in this area.
11. AAR research shows long term savings from reduced track structure damage and fuel consumption.
12. Controlling the impact load spectrum with WILD allows informed life decisions to be made for bridges and track components.

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HIGHWAY- RAILROAD GRADE CROSSING WARNING SYSTEM
DRAWN: L.O.I.(GEL)
DATE: 02/2/94
SHEET
3 OF 7
DWG NO. B-930917-3

DESIGN OF GATE DELAY AND GATE INTERVAL TIME FOR FOUR-QUADRANT GATE SYSTEM AT RAILROAD-HIGHWAY GRADE CROSSINGS

By: Fred Coleman, III, Ph.D.* and Young J. Moon**

Abstract

A design methodology for gate delay and gate interval time for at-grade crossings utilizing four-quadrant gates is developed. The design approach is based on the concept of dilemma zones related to signal change intervals at signalized intersections. The design approach is validated based on data from six sites in Illinois on a proposed High Speed Rail corridor. Gate delay and gate interval times are determined which provide an optimal safe decision point to allow a driver to stop before the crossing or proceed through the crossing without becoming trapped by the exit gates.

Introduction

The planning for introduction of new high speed (HS) rail passenger train service under Section 1010 of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) in the Chicago-St. Louis corridor involves, among many other activities, the review of grade crossing protection. The U.S. Department of Transportation (DOT) has established guidelines for grade crossing protection based on ranges of operating speeds (1). The Illinois Department of Transportation (IDOT) is proposing to operate high speed rail passenger service in this corridor at speeds of 125 mph (2). At these speeds the DOT guidelines require grade separation or the demonstration of new technologies which absolutely preclude entry into the crossings. To achieve the latter, IDOT has under consideration Four Quadrant Gates (QG) in conjunction with a Trapped Vehicle Detection (TVD) System. Quad gate installations have recently been placed in operating service on light rail and freight lines (3) to take advantage of their inherent ability to eliminate crossing violations after gate arms have been lowered.

Quad gate operation implementation issues can be initially divided into two categories: (a) gate operations; meaning operation of the gates related to timing of lowering entry (near) and exit (far) arms to allow vehicles to clear, and (b) constant warning times (CWT); meaning how should constant warning times be incorporated at crossings with QG serving freight and HS passenger trains.

This paper uses the analogy of a dilemma zone from research on traffic signal change intervals which assure that a high percentage of drivers can either clear the intersection or stop before entering. A railroad crossing adaptation of this concept is suggested to characterize driver behavior in response to initiation of flashing lights and activation of the gate descent. Operating data from six sites under consideration for four quadrant gates in the Chicago-St. Louis corridor are utilized to determine: 1) gate delay, the time interval after initiation of flashing lights, and 2) gate interval time, the time interval between entry and exit gate descent to assure that vehicles will have sufficient time to clear the crossing.

The long-term goal is to determine the operating time parameters which insure a safe system operation with the integration of quad gates into existing crossing safety systems such as Constant Warning Time (CWT) signals.

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QUAD GATE OPERATIONS

Function and Purpose

Quad gates are an additional pair of dual gate arms which are lowered on each side of a bi-directional crossing preventing any vehicle from crossing in-between the lowered gates, because their travel path is blocked on the front and rear side of the crossing. The implementation of quad gates complicates the crossing scenario because the likelihood of a vehicle clearing the crossing prior to exit gate descent must be taken into consideration. A primary concern is the possibility of a vehicle becoming "trapped" between the entry and exit gates. In addition, other issues arise in the operation of quad gates:

1) Introduction of a second gate arm (exit) prohibiting travel in the same direction may create driver indecision leading to a trapped vehicle incident,
2) The distance between multiple tracks will have an impact on exit gate timing,
3) Track crossing roughness may be a critical factor in some situations, and
4) Tractor-trailer and hazardous material vehicles in any combination of 1), 2), or 3) present potential for a worst case scenario for vehicle crossings.

Figure 1 is an example of a four quadrant gate crossing. Included in this figure are the areas on the approach to the crossing which define roadway lengths, where drivers are involved in the decision of whether to stop or to proceed through the crossing. The primary safety benefit from quad gates is that they assure no crossing violations after the gate arms are lowered, unless the gate arms are penetrated. The implementation of four quadrant gates, while eliminating gate arm violations, does present the potential for trapping a vehicle. Similar to a signalized at-grade intersection, vehicles approaching at various speeds must be allowed to clear the intersection during the yellow change interval. The at-grade crossing must, therefore, be analyzed relative to a change interval based on the ability of vehicles to clear the intersection. Driver behavior relative to stopping or proceeding at various speeds is the primary determinant of the likelihood of clearing the intersection.

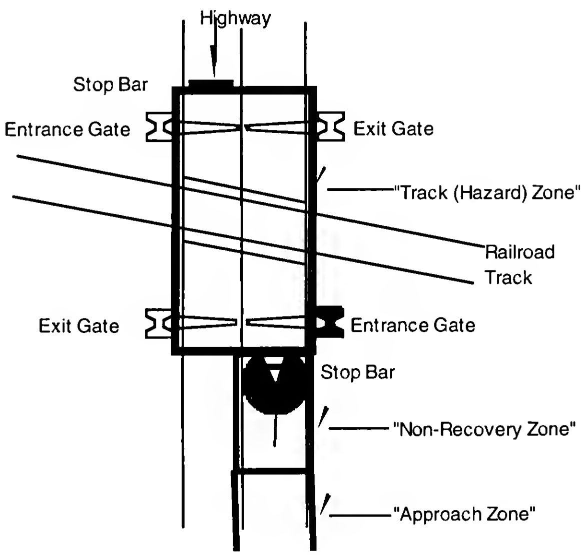

Figure 1. Quad gate system at railroad-highway grade crossing with single track.

Experience and Applications

Currently in the United States, there is limited experience with a QG system. Richards et al. (4) have obtained field trial results which involves crossings with freight train traffic. Their study sites had low to moderate vehicular volume with few trucks or vehicles required to stop before crossing. In the field trial, quad gates operating with flashing lights similar to standard gates, their effectiveness in reducing crossing violations after initiation of flashing lights was statistically significant at the 99 percent confidence level. For crossing violations after QG arms were down the effectiveness was statistically significant at the 99 percent confidence level.

Heathington et al. (5) identified six characteristics of crossings where quad gates (with skirts) would be good countermeasures to alleviate gate arm violations. These six characteristics are as follows:

- Crossings on four-lane undivided roadways;
- Crossings with two or more tracks separated by a distance equal to or greater than the storage requirements for one or more motor vehicles;
- Crossings with large variations in train speeds and without constant warning time;
- Crossings with consistent gate arm violations
- Crossings with continuing accident occurrences
- Crossings for which motor vehicle-train collisions pose large potential safety problems such as:
  - (a) crossings with a large number of hazardous materials trucks or trains carrying hazardous materials,
  - (b) crossings with a large number of school buses,
  - (c) crossings with high speed trains.

Three of the six characteristics were present at the six sites analyzed in this research:

- Crossings on four-lane undivided roadways;
- Crossings with two or more tracks separated by a distance equal to or greater than the storage requirements for one or more motor vehicles;
- Crossings for which motor vehicle-train collisions pose large potential safety problems such as:
  - (a) crossings with a large number of hazardous materials trucks or trains carrying hazardous materials,
  - (b) crossings with a large number of school buses,
  - (c) crossings with high speed trains.

Gate Operations

The literature (3, 5, 6) on quad gates has made suggestions for gate operations, with specific design guidelines suggested in 1993 by the Federal Railroad Administration (7). Gate operations are composed of two components as follows:

- Gate delay, defined as the time delay after the start of flashing signals and initiation of descent of the entry gate, and
- Gate interval, defined as the time delay after initiation of descent of the entry gate and initiation of descent of the exit gate.

If implementation of quad gates is to be undertaken, there needs to be design criteria, which assure the safety of motorists based on the operation of quad gates and the likelihood of clearing the crossing or stopping in an appropriate manner.

Gate Delay

Gate delay is five seconds for dual gate crossings in Illinois. For quad gates, no change in this operating policy is currently contemplated. The MUTCD (8) requires not less than 3 seconds for gate delay at grade crossings with dual gates after flashing signals commence. The Federal Railroad Administration (FRA) (7) suggested guidelines is three seconds based on its 1993 amendment to Emergency Order Number 15 with respect to application in Florida.

Gate Interval Time

In proposed guidelines issued in September 1993, the Federal Railroad Administration (7) has suggested that gate interval time (measured after initiation of entrance gate lowering) be 1 to 3 seconds. The New York State Department of Transportation (NYSDOT) in its High Speed Technology Demonstration Proposal (6) indicate an operating scenario which provides 10 to 12 seconds of gate interval time prior to initiation of descent of the exit gates.

DILEMMA ZONE CONCEPT WITH APPLICATION TO RAILROAD-HIGHWAY AT-GRADE CROSSINGS

History and Application

The dilemma zone concept refers to the research and methodology pioneered in traffic engineering related to drivers' decisions to stop or proceed at the onset of the yellow change interval. Sheffi and Mahmassani (9) state "The dilemma refers to the drivers' decision to proceed through the intersection or to stop when the signal indication changes from green to amber." The concept of a dilemma zone was recognized in the work of Gazis et al. (10), Olson and Rothery (11), Crawford (12) and Herman (13) and defined by Sheffi and Mahmassani (9) "as that zone within which the driver could neither come to a stop nor proceed through the intersection before the end of the amber phase." Continued work on this concept has led to a probabilistic approach to a driver stopping, with Zegeer (14) defining a dilemma zone as "the road segment where more than 10% and less than 90% of the drivers would choose to stop." Sheffi and Mahmassani (9) indicate "The approach consists of developing dilemma zone curves of "percent drivers stopping" versus "distance from stop bar" at the instant when the signal indication changes from green to amber."

Basis for Dilemma Zones at Railroad-Highway At-Grade Crossings

Drivers approaching an at-grade railroad crossing are faced with a similar scenario in which a visual signal in the form of flashing lights informs the driver of the need to stop, before the descent of the gates. The similarity to a signalized intersection is that, when drivers are some distance away from a crossing with gates when commencement of flashing lights occur, they must make a decision to stop or proceed. Therefore, determination of the zone boundaries for railroad-highway grade crossings based on "safe stopping distances" and the "clearance distance" given the approach speed and the width of the crossing using typical values of acceleration and deceleration rates are possible, similar to work performed by Gazis et al. (10).

Figure 2 shows the definition of dilemma and option zones which a driver faces when he/she is approaching an at-grade crossing.

In order to design the gate delay and gate interval time at railroad-highway grade crossings with quad gates, a dilemma zone is established in terms of the relationship between stopping distance (X_s), continuation distance (X_{con}), and clearance distance (X_{clr}). A driver approaching an at-grade crossing during gate delay, the time that the flashing light signals are operated before the entry gates are activated, will either have to stop or proceed to clear the crossing. Figure 3 shows the QG system of a crossing which includes all geometric data for calculating stopping distance as well as clearance distance.

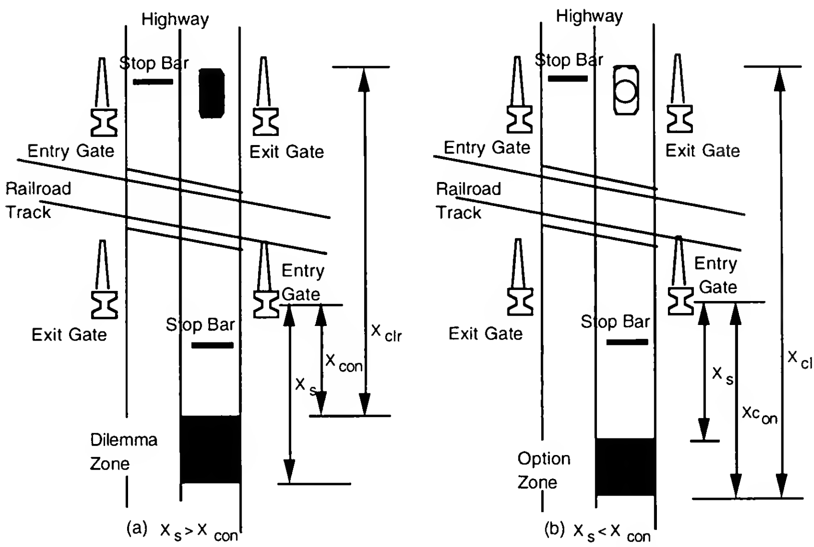

Figure 2. Definition of dilemma and option zones; (a) Dilemma Zone; (b) Option Zone.

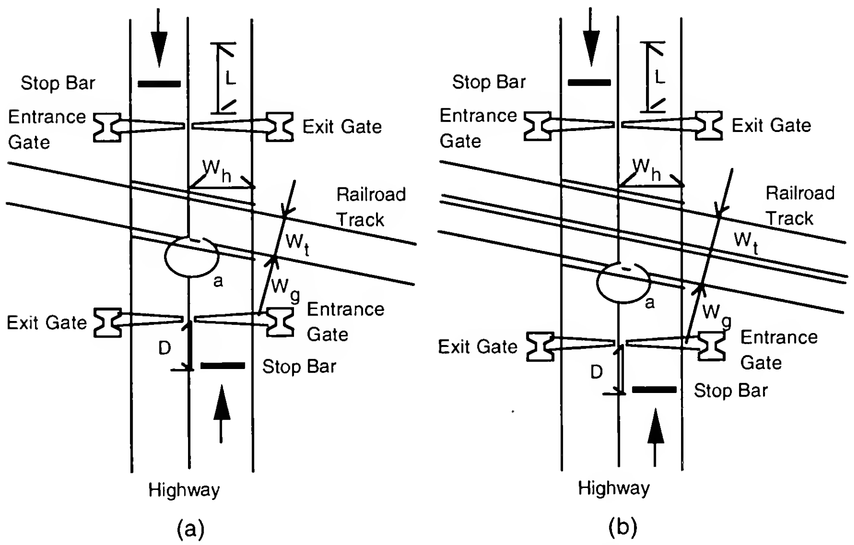

Figure 3. Geometric data for calculating stopping distance and clearance distance; (a) single railroad track; (b) multiple railroad tracks.

The stopping distance is the distance required for vehicles to stop at the stop bar usually 6–8 ft in front of the gate arm. This stopping distance for an at-grade crossing is the same for signalized intersections, except it includes the distance between stop bar and gates. It is formulated as follows:

$$X_s = tv + \frac{v^2}{2(a + G \cdot g)} + D \tag{1}$$

where,

X_s = stopping distance (ft);

t = driver perception-reaction time (PRT) (sec);

v = approach speed (ft/sec);

a = deceleration rate on level pavement (ft/sec2);

G = acceleration due to gravity (ft/sec2);

g = grade of approach lanes (percent/100); and

D = distance between stop bar and gates (ft).

The continuation distance is defined as the distance a vehicle travels during initiation and/or the interval of gate delay prior to descent of the entry gate at the end of the dilemma zone (or somewhere in the option zone), with no change in speed that results in the vehicle position perpendicular to the entry gate. Figure 2 indicates where the continuation distance, X_{con}, could occur.

The clearance distance is defined as the distance a vehicle travels to clear the crossing through the exit gates, including vehicle length. The clearance distance is a function of the distance within which the vehicle can clear the crossing, before the end of the quad gate operation time which consists of gate delay, plus gate interval time. Figure 2 presents the clearance distance, X_{clr}, in relation to the stopping distance, X_s, and continuation distance, X_{con}.

Mathematical Relationships between Continuation Distance, and Clearance Distance

Referring to Figure 2 and the dilemma zone concept, if a vehicle were to proceed to clear the crossing during the quad gate operation time, the vehicle would cover a clearance distance which consists of: (1) the continuation distance, (2) the distance between entry and exit gates, and (3) the length of the vehicle. Figure 3 shows the QG system of a crossing which includes all geometric data for calculating stopping distance, as well as clearance distance. Considering the approach speed, the clearance distance required for a vehicle to clear the crossing is formulated as follows:

$$X_{clr} = T_G \cdot \mathrm{v} \tag{2}$$

where,

X_{clr} = clearance distance (ft);

T_G = quad gate operation time (sec); and

v = approach speed (ft/sec).

While T_G gives us the total gate operation time, it is also the total travel time for the vehicle to clear the crossing. This total travel time contains both a gate delay and a gate interval time component. The gate delay time component contains the vehicle travel time for the continuation distance, which effectively places the vehicle at the entry gate prior to entry gate descent. The gate interval time component contains the travel time beginning with gate descent through the crossing geometry, which includes the distance between entry and exit gates, as well as the length of vehicle to clear the exit gates. Referring to Figure 3, considering the crossing angle between the railroad and highway, as well as road segments immediately adjacent to the railroad track, the distance between entry and exit gates is formulated as follows:

$$W_{ght} = \frac{W_t}{\sin\alpha} + \frac{2 \cdot W_h}{\tan\alpha} + \frac{2 \cdot W_g}{\sin\alpha}, \ \alpha \le 90^\circ$$
$$= \frac{W_t}{\sin(180-\alpha)} + \frac{2 \cdot W_h}{\tan(180-\alpha)} + \frac{2 \cdot W_g}{\sin(180-\alpha)}, \ \alpha > 90^\circ \qquad (3)$$

where,

W_{ght} = distance between entry and exit gates (ft);

W_t = width of railroad track (ft);

W_h = width of approaching lane of the highway (ft);

W_g = distance from track edge to gate (ft);

α = crossing angle (deg).

Then, the continuation distance required for a vehicle to clear the crossing can be formulated as follows:

$$X_{con} = X_{clr} - (W_{ght} + L) \qquad (4)$$

where,

X_{con} = continuation distance (ft);

X_{clr} = clearance distance (ft);

W_{ght} = distance between entry and exit gates (ft);

L = length of the vehicle (ft).

As shown in Figure 2a, if a vehicle approaches a crossing during gate delay and if $X_s > X_{con}$ and the vehicle is positioned between X_s and X_{con} such that $X_s > X > X_{con}$, a dilemma zone exists where a vehicle could neither stop nor clear the crossing. Referring to Figure 2b, if $X_s < X_{con}$ and the vehicle is positioned between X_s and X_{con} such that $X_s < X < X_{con}$, an option zone exists for which a driver can choose between stopping and clearing the crossing. If $X_s = X_{con}$, the dilemma and option zones are eliminated and a point or distance is obtained which assures the likelihood of drivers stopping or clearing the crossing. This distance is called a "Safe Decision Location."

Design of Gate Delay and Gate Interval Time

When $X_s = X_{con}$, the assumption is that the two distances are equal, meaning that the point where this occurs eliminates the option zone and the dilemma zone. More importantly, the driver is made aware of this distance they must cover by the activation of the flashing signals, which is the onset of gate delay. Based on the "Safe Decision Location," where $X_s = X_c$, the simplification results as follows:

$$t \cdot v + \frac{v^2}{2(a + G \cdot g)} + D = v \cdot T_G - (W_{ght} + L)$$
$$t \cdot v + \frac{v^2}{2(a + G \cdot g)} + D + (W_{ght} + L) = v \cdot T_G \qquad (5)$$

This gives

$$T_G = \left\{ t + \frac{v}{2(a + G \cdot g)} + \frac{D}{v} \right\} + \left\{ \frac{1}{v}(W_{ght} + L) \right\}. \qquad (6)$$

The determination of an equal distance for stopping or continuing connotes that the driver has an opportunity to stop, or an opportunity to continue up to the entry gate through an optimal derivation of gate delay time, because when $X_s = X_{con}$ and the total gate operation time (T_G) is determined, the first term is the optimal gate delay time, while the second is the optimal gate interval time for the vehicle to clear with an assumption of constant speed. From Equation (6),

$$T_G = T_D + T_I,$$
$$T_D = \left\{ t = \frac{v}{2(a + G \cdot g)} + \frac{D}{v} \right\}, \tag{7}$$
$$T_I = \left\{ \frac{1}{v}(W_{ght} + L) \right\}$$

where,

T_G = gate operation time (sec);

T_D = gate delay (sec); and

T_I = gate interval time (sec).

The gate delay (T_D) is independent of the crossing angle, however, the gate interval time (T_I) must include the crossing angle. Gate delay is based on human factors and driver behavior. Using approach speeds, t (driver perception-reaction time, PRT) value of 1.0 sec and 2.5 sec, and varying the deceleration rate, Figure 4 presents the gate delay requirements. With a deceleration rate of 10 ft/sec^2 similar to the intersection studies with an approach speed of 35 mph, 3.7 sec. of gate delay is required. As a whole, the gate delay should be approximately 3.0–4.0 sec at t (PRT) = 1 sec, or 4.5–5.5 sec at t (PRT) = 2.5 sec based on AASHTO (16). However, slightly longer times may be justified if vehicle approach speeds are over 40 mph.

Figure 5 shows the gate interval time requirements based on the second term in Equation 6 in terms of speed in the track zone. This gate interval time is the total gate interval time available. If the interval is assumed to begin at the start of entry gate descent, and a vehicle at the entry gate decides to clear the crossing, only 2–3 seconds is available before the gate would come into contact with the vehicle. Utilization of this 2–3 seconds does not diminish the total gate interval time for this vehicle, since the same amount of time to avoid contact with the exit gate is available.

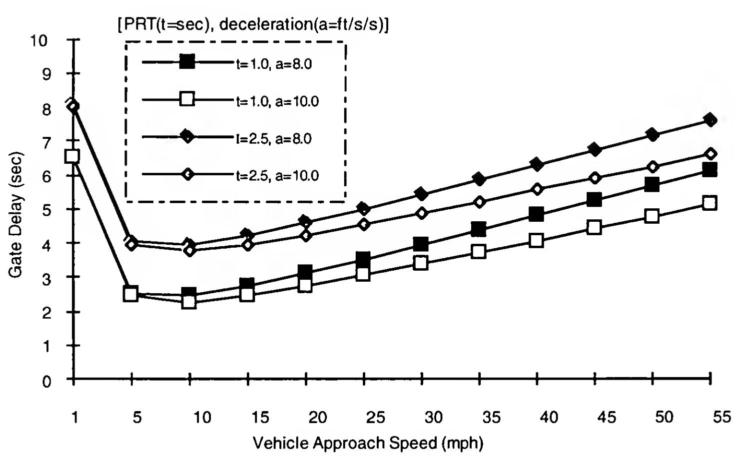

Figure 4. Gate delay required as a function of vehicle approach speed and deceleration rate.

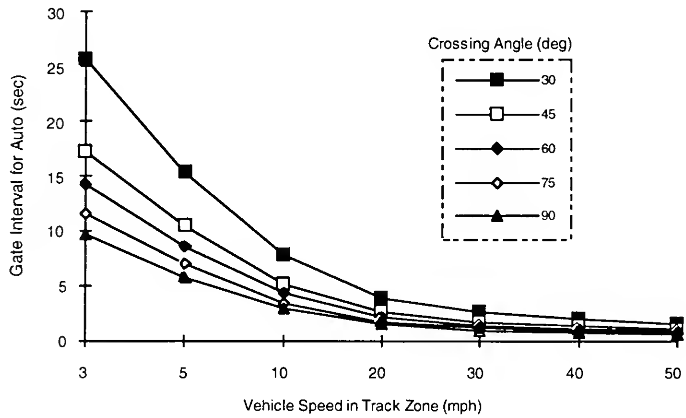

Figure 5. Gate interval time requirements as a function of vehicle speed in track zone and the angle of crossing.

In assessing the criteria for gate interval time, the ability of a vehicle to clear the railroad track zone under low speed conditions was felt to be the primary consideration from a safety standpoint. Figure 5 indicates that for autos, vehicle speed in the track zone (see Figure 1) is the primary factor in determining gate interval time. At speeds less than 10 mph, gate interval time increases sharply, while at speeds greater than 10 mph, approximately 5 seconds is sufficient. This finding indicates that crossing speed as determined by grade crossing roughness, and driver behavior such as looking, precautionary slowing, preceding vehicular clues or traffic lights should be recognized as one of the primary considerations in the determination of gate interval time. However, if the gate interval is excessive in length the credibility of the quad gate system to drivers will be lacking.

Concept Validation Based On Field Data

In order to determine both gate delay and gate interval time for actual sites, six crossings under consideration for four quadrant gates in Illinois were evaluated. Each site's operating and geometric data are shown in Table 1.

Table 2 shows the gate delay needs at the six sites utilizing a deceleration rate of 10 ft/sec^2. Gate delay with 1 second of PRT is approximately 4 seconds at an approach speed of 35 mph and 5 seconds with an approach speed of 45 mph. For 2.5 seconds of PRT, approximately 5 seconds of gate delay at 35 mph is required and 6 seconds at 45 mph. Gate delay is independent of the crossing angle and depends only on the approach speed and deceleration rate, as indicated in the first term of Equation (7).

Table 3 indicates the gate interval time needs at these sites. Five of the crossings are not at a right angle. In addition, the Gardner and Pontiac site crossing surfaces have been evaluated as "rough" (15), suggesting that minimum vehicle speeds are appropriate. All other sites evaluated as "good" have assumed crossing speeds of 5 mph. All the crossings are level (0% grade).

Utilizing the geometry and assumed minimum speed on the track zone, the overall gate interval time varies from approximately 7–15 seconds for a passenger vehicle. For a WB-60 truck defined by AASHTO (16) which has 65 ft. of vehicle length, approximately 14–22 seconds of gate interval time would be required for clearing the vehicle safely at the crossing. It should be noted that the

Table 1. Operational and Geometric Data or Selected Sites in Illinois

Location of Crossing (City Name in Illinois)	McLean	Springfield	Hartford	Gardner	Pontiac	Chenoa
Veh. Approach Speed (85th-Percentile) (mph)	45	35	40	35	25	25
Roughness	Good	Good	Good	Rough	Rough	Good
Assumed Min. Veh. Speed in Track Zone, v_t (mph)	5	5	5	3	3	5
Angle, α (Deg)	85	70	95	80	90	80
% Heavy Truck	16	3	0	5	0	5
Train Type	HS / Freight	Freight	HS / Freight	HS / Freight	HS / Freight	HS / Freight
W_t (ft)	20	5	55	5	5	5
W_g (ft)	12	22	15	12	14	12
W_h (ft)	11	30	18	10	12	9
D (ft)	8	8	8	8	8	8

Table 2. Gate Delay for 6 Crossings in Illinois

Crossing Street, City	Approach Speed (mph)	Gate Delay at *t* (PRT) = 1	Gate Delay at *t* (PRT) = 2.5
U.S. Route 136, McLean	45	4.5	6.0
N. Grand Ave., Springfield	35	3.7	5.2
Hawthorn St., Hartford	40	4.1	5.6
Main St., Gardner	35	3.7	5.2
Main St., Pontiac	25	3.1	4.6
Trunk Route 35A, Chenoa	25	3.1	4.6

Table 3. Gate Interval Time for 6 Crossings in Illinois

Crossing Street, City	Min. Speed on Track Z. (mph)	Gate Interval for Auto	Gate Interval for Truck
U.S. Route 136, McLean	5	8.9	15.1
N. Grand Ave., Springfield	5	14.3	20.5
Hawthorn St., Hartford	5	14.7	N.A.[a]
Main St., Gardner	3	11.4	21.9
Main St., Pontiac	3	11.8	N.A.[a]
Trunk Route 35A, Chenoa	5	7.0	13.3

[a]No heavy truck volume reported at these sites.

major factors which influence the gate interval time are the minimum vehicle speed in the track zone, width of the crossing (i.e. distance between entrance and exit gate), the length of the vehicle, and the angle of the crossing.

Findings

Table 4 and Table 5 present the input values and results to determine the Safe Decision Location ($X_s = X_{con}$) distance using the approach speed and then the minimum speed in the track zone for automobiles.

The difference between the clearance distance and stopping distance is computed for both assumed speeds in the track zone. The comparison of the difference between Safe Decision Location using approach speed and minimum speed on the track zone determines if the speeds in this area require a different gate interval time. The results in the last column of Table 4 and Table 5 indicate that the differences in the Safe Decision Location are insignificant and, therefore, the gate interval time computed for the minimum speed in the track zone is adequate and does not provide excessive gate interval time for vehicles crossing the track at normal speed.

Conclusions

The approach suggested along with the findings presented utilizing site data from crossings in Illinois indicate that utilizing the concept of a dilemma zone provides a design procedure for gate delay and gate interval times. Further, it is demonstrated that by incorporating the geometry of a site, speed of approach, width of crossing, and minimum assumed speed in the track zone, a sufficient gate interval time for automobiles could be provided. This overall design approach was validated through comparison of the Safe Decision Location and was found to provide essentially the same distance for a driver to stop or continue through the crossing, since the gate operation time included the necessary gate interval time to allow clearance of the crossing.

Table 4. Validation of Safe Decision Location for 6 Crossings (PRT = 1.0 sec)

Crossings	Speed[a] (mph)	T_D (sec)	T_I (sec)	T_G (sec)	X_{clr} (ft)	X_{con} (ft)	X_s (ft)	$X_{con}-X_s$ (ft)
U.S. Route 136,	v=45, v_t=45	4.50	0.98	5.48	362.59[b]	293.59	292.94[d]	0.64
McLean	v=45, v_t=5	4.50	8.90	13.40	363.09[c]	292.09	290.24[e]	3.85
N. Grand Ave.,	v=35, v_t=35	3.70	2.03	5.73	294.56[b]	190.37	191.81[d]	–1.44
Springfield	v=35, v_t=5	3.70	14.30	18.00	295.47[c]	191.28	189.10[e]	2.18
Main St.,	v=35, v_t=35	3.70	0.98	4.68	240.58[b]	190.37	191.81[d]	–1.44
Gardner	v=35, v_t=3	3.70	11.40	15.10	240.64[c]	190.43	190.83[e]	–0.40
Main St.,	v=25, v_t=25	3.10	1.41	4.51	165.93[b]	113.93	112.28[d]	1.65
Pontiac	v=25, v_t=3	3.10	11.80	14.90	165.96[c]	113.96	111.31[e]	2.66
Trunk Rt. 35A,	v=25, v_t=25	3.10	1.40	4.50	165.55b	113.93	112.28[d]	1.65
Chenoa	v=25, v_t=5	3.10	7.00	10.10	165.38c	113.76	109.58[e]	4.18

[a] v = approach speed, v_t = speed in the track zone.

[b] Clearance Distance = $T_G \cdot v$ [by Equation (2)].

[c] Clearance Distance = $T_D \cdot v + T_I \cdot v_t$

[d] $X_s = v \cdot t + v^2 / (2 \cdot a) + D$ [by Equation (1) and t(PRT) = 1 sec].

[e] $X_s = v \cdot t + (v^2 - v_t^2) / (2 \cdot a) + D$ [t(PRT) =1 sec].

Table 5. Validation of Safe Decision Location for 6 Crossings (PRT = 2.5 SEC)

Crossings	Speed[a] (mph)	T_D (sec)	T_I (sec)	T_G (sec)	X_{clr} (ft)	X_{con} (ft)	X_s (ft)	$X_{con}-X_s$ (ft)
U.S. Route 136,	v=45, v_t=45	6.00	0.98	6.98	461.73[b]	392.73	392.17[d]	0.56
McLean	v=45, v_t=5	6.00	8.90	14.90	462.32[c]	393.09	389.47[e]	3.62
N. Grand Ave.,	v=35, v_t=35	5.20	5.20	7.23	371.98[b]	267.79	268.99[d]	−1.20
Springfield	v=35, v_t=5	5.20	14.30	19.50	372.65[c]	268.46	266.28[e]	2.18
Hawthorn St.,	v=40, v_t=40	5.60	1.83	7.43	436.88[b]	329.41	327.87[d]	1.54
Hartford	v=40, v_t=5	5.60	14.70	20.30	437.33[c]	329.86	325.17[e]	4.69
Main St.,	v=35, v_t=35	5.20	0.98	6.18	317.96[b]	267.75	268.99[d]	−1.24
Gardner	v=35, v_t=3	5.20	11.40	16.60	317.81[c]	267.60	268.01[e]	−0.41
Main St.,	v=25, v_t=25	4.60	1.41	6.01	220.87[b]	168.87	167.41[d]	1.46
Pontiac	v=25, v_t=3	4.60	11.80	16.40	221.09[c]	169.09	166.44[e]	2.65
Twp. Rd. 35A	v=25, v_t=25	4.60	1.40	6.00	220.50[b]	168.88	167.41[d]	1.47
Chenoa	v=25, v_t=5	4.60	7.00	11.60	220.50[c]	168.88	164.71[e]	4.17

[a] v = approach speed, v_t = speed in the track zone.
[b] Clearance Distance = $T_G \cdot v$ [by Equation (2)].
[c] Clearance Distance = $T_D \cdot v + T_I \cdot v_t$
[d] $X_S = v \cdot t + v^2 / (2 \cdot a) + D$ [by Equation (1) and t (PRT) = 2.5 sec].
[e] $X_S\ v \cdot t + (v^2 - v_t^2) / (2 \cdot a) + D$ [t (PRT) = 2.5 sec].

Acknowledgments

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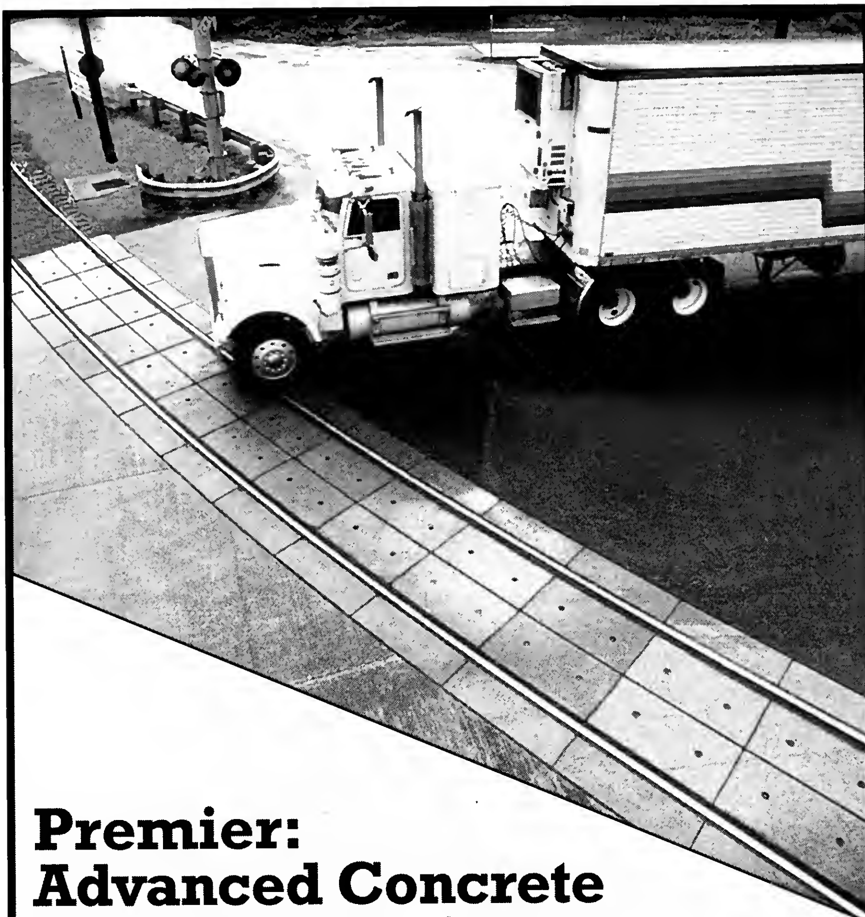

BRIDGE CONSTRUCTION IN RESPONSE TO HEAVY AXLE LOADS UPRR-JOPPA SUBDIVISION

By: Donald L. Steele* and Tom Skinner**

Introduction

In early 1993, the Union Pacific Railroad decided to upgrade its Joppa Subdivision in southern Illinois. The Union Pacific's Coal Marketing Department had successfully negotiated a coal contract requiring heavier unit trains to begin service over the Joppa Subdivision in October of 1994. These heavier coal trains originate in the Powder River Basin, in Wyoming, and travel to an unloading facility on the Ohio River where coal is reloaded onto barges and shipped to power plants upriver.

The primary focus of the Joppa Subdivision upgrade was the replacement of two large railroad bridges; the 845' 8" long Little Saline Creek Bridge and the 650' 8" long Grasshopper Creek Bridge. This paper presents the story of how these two 1899 vintage trestles were safely constructed in 18 months, under traffic, under budget, and ahead of schedule.

The two 1899 vintage steel frame trestles are located near Goreville, Illinois, which is approximately 20 miles southeast of Carbondale, Illinois. The Little Saline Creek Bridge is an open deck trestle having 23 spans (see Figure 1). The superstructure is composed of twelve 47' 6" long drop-in deck truss spans and eleven beam spans (one beam span at each of the eleven steel frame towers). The Grasshopper Creek Bridge is of similar construction, with 17 spans (see Figure 2). The superstructure is composed of ten 47' 6" long drop-in deck truss spans and seven 25' 0" long beam spans which are identical to the Little Saline Creek Bridge.

The piers for each bridge are concrete pedestals, which are spread footings on soil. The abutments are large, gravity type stone masonry abutments on spread footings. The old abutments were refurbished several years ago, with reinforced concrete to "jacket" around and strengthen the upper portion of the abutment breastwall and backwall.

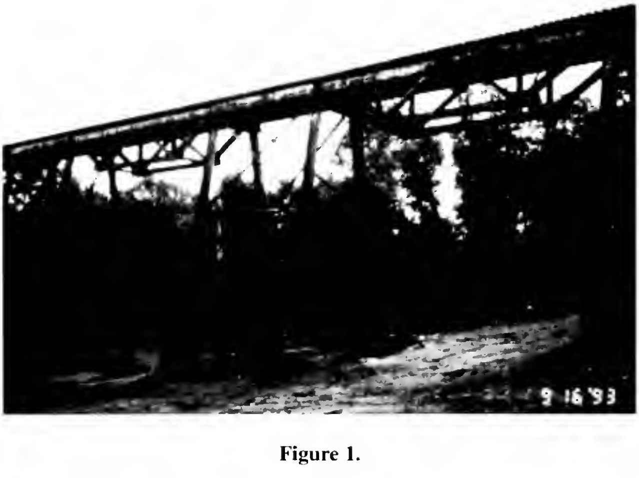

Figure 1.

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Figure 2.

In March of 1993, the Union Pacific Railroad requested bids from Consulting Engineering Firms for conceptual design and proposed schedules to construct (under traffic) the Little Saline and Grasshopper Creek Bridges. Many good ideas were returned. HNTB Corporation in Kansas City, Missouri was retained to design the bridge replacements. HNTB's responsibilities were to perform the design surveys, prepare the final design plans and specifications, and to develop a span "change-out" procedure that could accommodate a maximum window (no train operations) of 12 hours. Daily work windows where limited to 6 hours to meet joint line operations with the Union Pacific Railroad and the Burlington Northern Railroad traffic schedules. HNTB completed the work by developing 122 plan drawings and the associated specifications in 93 calendar days. When the final design was completed, the project was advertised for bidding.

Because delivery of the structural steel was a project critical path item, a separate contract for fabrication and delivery of the structural steel was advertised in June of 1993. The remainder of the project which included the construction of the piers, abutments, temporary jump spans, erection of the steel, installation of the ballast pans, raising the bridge to new grade and site clean-up was advertised in July of 1993. The key suppliers for the project were:

General Contractor	Walsh Construction of Chicago
Designer	HNTB Corporation of Kansas City
Structural Steel Fabricator	Stupp Brothers of St. Louis
Specialty Contractors:	
Drill Shaft Installation	Case Foundation of Roselle, IL
Precast Concrete Ballast Deck Pans	Wilson Concrete of Ohama
Steel Erector	Rednoir of Cutler, IL

Bridge Design

The design of the new bridge structures was for a ballasted deck with E 80 live load with diesel impact. The design also included provisions for seismic forces. In 1993, AREA contained no criteria for seismic design and because AREA Committee 9, Seismic Design, was still not organized, HNTB used the AASHTO Seismic Performance Category B for the Seismic Design Criteria. The coefficient of gravitational acceleration was 0.15 G. This was based on the USGS 250 year event specifically stated as a 10% probability of occurrence in 50 years.

A single mode analysis was not only performed for transverse loadings, but for longitudinal loading assuming simple spans, and longitudinal loading assuming continuous spans. The longitudinal continuity was assumed to be developed by the ballast and the continuous welded rail on the structure.

Three additional design issues also had to be addressed as follows:

1. Because the existing masonry bridge foundation pedestals were spread footings on soil, the new pier foundations had to be constructed in such a manner as to eliminate any possible disturbance or settlement of the existing masonry pedestals.
2. The existing spans had to be changed-out one or two at a time, because a maximum of 12 hours of track time was all that was allowed.
3. The existing stone abutments were to be replaced due to their age and uncertain performance under the proposed heavier loads.

To solve the issue of foundation stability, it was decided not to use traditional pile supported footings where open excavations could be an invitation to settlement or sliding of the adjacent spread footings. Instead, it was decided to use drilled shaft foundations which could be lined with steel casings to retain the stability of the surrounding soil and maintain the integrity of the existing substructure during construction.

The issue of span change-out was solved by strategically placing the new piers directly under one of the panel points of the drop-in truss span. First, this permitted the contractor to completely remove the taller towers during the span change-out sequence. It also provided a convenient way to salvage and re-use a portion of the drop-in truss spans, thus eliminating the need for extensive and expensive temporary jump spans during each change-out.

The change-out was also facilitated by erecting the new spans on cantilever brackets supported by the new pier caps. By utilizing re-usable falsework and jacking frames attached to the top of each pier, a cost savings over ground supported falsework was realized. This span change-out approach also ensured that during each operation, the new spans could be placed immediately after the old spans were removed. All the contractor had to do was slide the new girders into position, using a pair of small winches, which were also mounted to the top of the pier.

The existing masonry abutments were replaced by constructing new abutments behind the old abutments. Temporary jump spans were designed along with a temporary steel end bent, and the new concrete abutment was then constructed under traffic, behind the existing abutments.

Description of New Bridges

The new bridge over Little Saline Creek is 878' 11" long (see Figure 3). The superstructure is composed of ten ballasted deck plate girder spans ranging in length from 51' 0" to 121' 9". The girder depth varies, from 84 inches to 120 inches. The precast prestressed concrete ballast deck pan are 16' 6" out to out and 4' 6" long.

The nine substructure piers are composed of twin 8 foot diameter column concrete piers, which are in turn founded on eight foot diameter drilled shafts, which are socketed 20 feet into rock (see Figure 4). The columns are typically reinforced with forty-six No. 11 bars vertically and horizontally with No. 5 ties varying from 4 inches to 12 inches on centers. The seismic cross ties are No. 5 bars placed at 4 inch centers, in an interlocking grid pattern.

The substructure capbeams are 8' 0" deep and 9' 0" wide. The bottom longitudinal reinforcement is composed of twenty-four No. 11 bars with triple No. 7 stirrup bars for shear transfer.

The new bridge over Grasshopper Creek is 693' 5" long (see Figure 5) and is similar to the Little Saline Creek Bridge.

Figure 3.

Figure 4.

Construction Schedule

The contract for fabrication delivery for the structural steel was awarded to Stupp Brothers in July of 1993. The General Contractor produced a project schedule calling for steel fabrication and delivery to be on a "right on time basis." From the schedule it can be noted that construction of the concrete substructure began in September of 1993. Actual field work began on September 4, 1993, with the clearing and grubbing and the beginning of the installation of the drilled shafts at the Little Saline Creek.

Drilled Shaft Construction

The sequence of drilled shaft construction was to first use an 8' 0" diameter flight auger. Next, a temporary casing 9' 0" in diameter was set into the perimeter of the excavated hole to stabilize the

Figure 5.

top 6 to 10 feet of earth and protect the existing footings from detrimental movements. The temporary casing was incrementally advanced into the ground. First, the 8' 0" diameter flight auger was used to remove 18 inches to 24 inches of overburden. The temporary casing was then turned into the hole. This reiterative process continued until the temporary casing was extended below the existing footings.

With the temporary casing in place, the smaller 8' 0" diameter permanent casing was aligned and then telescoped into the hole and advanced to founding elevation, by alternately using the auger to remove the overburden and turning the permanent casing into the ground. The permanent casing was advanced until it seated at top of rock.

When top of rock was reached, the augers were replaced with core barrels ranging in size from 2 feet to 8 feet in diameter. The shaft was cored to founding elevation using the two foot diameter core barrel, which also served as pilot hole for future drilling. The large screw auger was used to remove rock, when the rock broke up into smaller fragments and could not be removed by the core barrels. The process of excavating the drilled shaft continued by using the smaller and then progressively larger core barrels, until the foundations penetrated 20 feet into solid rock.

Once the drilled shaft had advanced to the founding elevation, a 20 foot long by 4 inch diameter rock core was taken at the bottom of the selected shafts to ensure that the drilled shaft would have adequate end bearing on solid rock. With proper end bearing confirmed, the contractor would lift the preassembled reinforcing bar cage into position and carefully lower it into the excavation. Once the reinforcing bar cage was in proper position, the concrete was placed in the drilled shaft using a tremie tube and pumping truck. After the concrete cured, the temporary casing was removed and the small void around the permanent casing was filled with sand.

In most areas, the drilled shafts were dry and the surrounding materials could easily be stabilized using eight foot diameter permanent steel casing. In some cases, the shafts would fill with water and neither the temporary casing nor the permanent casing could be advanced quickly enough or deep enough to stabilize the surrounding materials. In these few cases, a benonite slurry was pumped into the excavation to provide a equalizing hydraulic pressure and stabilize the surrounding soil material. The benonite slurry was reclaimed as the drilled shaft was pumped full with concrete using a concrete pump and tremie tube. Special precautions were taken at this time to protect the work area and surrounding environment from any damage due to spills and material overflows.

Reinforcing Bar Cages

While the drilled shaft operation was in progress, the contractor was building the reinforcing bar cages in a "jig," fabricated at the site. The reinforcing bar cages are 7' 6" in diameter and were assembled in one piece to extend from the bottom of the drilled shaft to one bar lap length above the top of the permanent casing. The drilled shaft reinforcing cages ranged in length from 30 feet to over 80 feet. The reinforcing bar cages were so large that the iron workers found that it was easier to work from inside the reinforcing cage in lieu of trying to reach between the bars from the outside. This permitted four iron workers to build the bar cages by having two workers on either side of the reinforcing bar mats.

Column Construction

The contractor used specially made, project specific 8' 0" foot diameter steel forms for the construction of the columns. The circular steel forms were composed of two half round forms, which bolted together in various lengths and easily adjusted to the varying length columns. The column construction progressed by first erecting the preassembled reinforcing bar cage and then sliding the steel column forms down over the reinforcing steel. The forms were secured in position and filled with concrete using the tremie tubes and a pumper truck.

Capbeam Construction

Once the column concrete reached sufficient strength, the column forms were removed and the capbeam construction began. First the side forms for the cap were placed. Because the existing structure prevented the bar cage for the capbeam from being preassembled and lowered into the forms, the contractor used a rebar frame to assist in the placement of the reinforcing steel for the capbeams. The longitudinal bars were placed on the frame and hoisted into position at the end of the capbeam. The iron workers then just pulled the bar across the frame and slid them into the appropriate location in the cap.

Once the reinforcing bar cage was tied together, the ends of the capbeam forms were placed. The anchor bolts for the future spans and temporary spans were secured in position with templates, and after a final check, the capbeam was cast, typically using a concrete pumper truck.

Two key elements considered during the capbeam construction were the location of the bearing device anchor bolts and the projection of the anchor bolts above the top of the cap beam. If the bolts projected too high, the new spans could not be translated across the top of the pier and into their final position. These anchor bolt clearances were rigorously checked to ensure future problems would not occur during the critical 12 hour span change-out period.

Temporary Jump Spans

Construction of the new steel H pile supported abutments was accomplished by first installing a 58 foot temporary jump span behind the existing masonry abutments. Steel piling for the temporary abutment and permanent abutment was driven during the six hour work window the contractor was provided on a daily basis. During a twelve hour track closure, the existing ballast and embankment behind the old masonry abutment were removed; a portion of the old masonry backwall was removed; the steel piling at the temporary abutment was cut off and a steel cap was placed; and a 3/8 inch steel plate was placed behind the temporary steel piles to act as a temporary retaining wall for the approach track embankment fill material. With this preparatory work completed, the prefabricated preassembled temporary steel jump span was placed in position on the old masonry abutment and the temporary pile abutment. The ties and rail were placed on the new span and a safety inspection made prior to opening the track to rail traffic.

New Abutment Construction

Once the temporary jump spans were in place, the new abutment construction proceeded in a conventional manner. First, the footing excavation was completed and then the permanent steel piles were cut off at the correct elevation. The reinforcing steel was placed and concrete forms were set. The concrete was placed using a concrete pumper truck. By late January 1994, the substructure for the Little Saline Creek Bridge neared completion and the contractor moved the substructure construction activities to the Grasshopper Creek Bridge and began preparing for the span change-out on the Little Saline Creek Bridge.

Span Change-Out

Preparation for span change-out involved numerous activities. First, the jacking frames and falsework support brackets had to be erected at the top of the new piers (see Figures 6 & 7). The sliding beams and winches were installed and tested. The more complex work began when the structural steel girders arrived by truck from Stupp Brothers. The contractor simply picked the girders off the truck and set them on the falsework brackets. The lateral bracing, crossframes and inspection walkway were installed and checked for proper fit. The concrete ballast deck pans were placed, along with the safety handrails. The waterproofing was placed and final checks made to ensure all details were addressed.

On the day each span change-out occurred, a carefully orchestrated and sequenced plan of events was followed. Rail traffic was interrupted at 7 a.m. The Union Pacific Railroad forces cut and removed the rail at the designated locations. The railroad ties were removed by the contractor. The drop-in truss spans were cut free of the towers using a torch to cut 16 rivets, and the spans were lifted out using a crane. The towers were then cut free at the top of the masonry pedestals and simply pulled over sideways by a crane until they fell to the ground.

The falsework skid beams were lubricated using common dishwashing detergent between the stainless steel sliding plates and Teflon surfaces. One winch at each pier was operated simultaneously to slide the new span into position. This sliding of the spans took about two minutes. Track panels were lifted into place directly on the waterproofing and secured to the edge of the concrete pans by using timber struts. After a safety review, the track was opened to service, after as little as 10 hours out of service. Ballast was not placed immediately to minimize the bridge dead weight to facilitate the planned track raise.

Shim packs were used to raise the girders to accommodate for a 21 inch track raise. This was accomplished by taking raises in small steps, so a sharp grade change did not occur. This track raise was accomplished over an eight day period. A deliberate process, but it worked. The shims within the packs ranged from 1/4 inch to 3 inches thick. Three spans were raised in a given day. One span was brought to proper elevation and the next two were brought to differential elevations, so proper track runoff could occur. Track shims were used for final grade adjustment. When the bridge girders were

Figure 6.

Figure 7.

in proper elevation, ballast was dumped and the track was adjusted to final grade and position. The only thing left was the placing of the CWR, final cleanup and touching up of the area.

Points of Interest

Complex projects can be undertaken and successfully completed in a safe environment. There was only one report of an injury and that was a pinched finger. Safety was up front and foremost throughout the project.

Planning must be a joint effort of the designer, contractor and the railroad to ensure the project is visualized and conceptualized by addressing all safety, operational, construction and design related issues, and that the project objectives are kept in focus at all times.

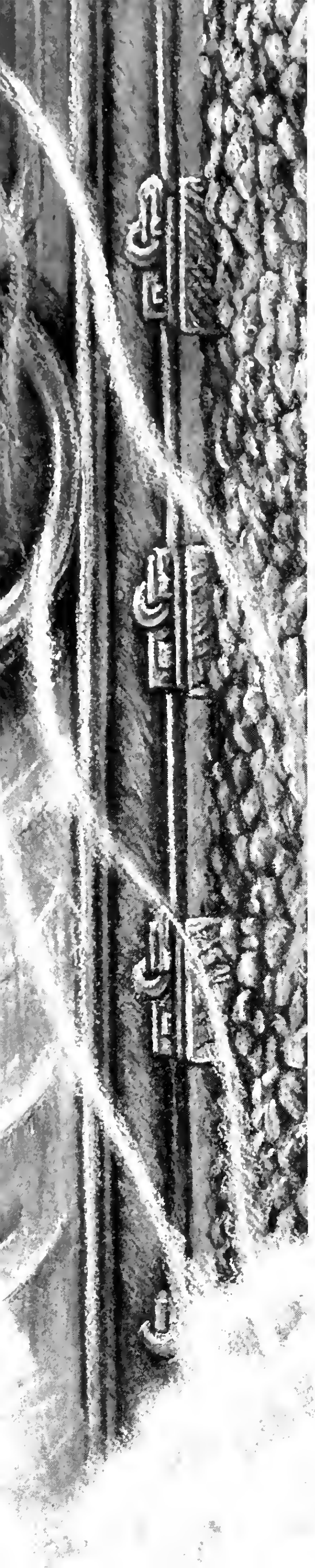
Additionally, systematic review of the project plan and schedule is mandatory to verify that the project schedule, cost and quality are continuously addressed.

A check list of events during critical operations helped verify all issues.

The following materials went into the construction of the project.

Description	Little Saline	Grasshopper
Length	878.92 LF	690.75 LF
Bridge No. of Spans	10 Ea.	8 Ea.
Steel H Pile	1093 LF	924 LF
Concrete	1860 CY	1957 CY
Excavation	2200 CY	2200 CY
Reinforcing Steel	883238 LB	755105 LB
Drilled Shafts	934 LF	585 LF
Permanent Casing	355 LF	275 LF
Structural Steel	1360100 LB	943255 LB
Shims	49.88 FT	27.48 FT

The project is a true success story. It was completed under budget, three months ahead of schedule and with only one small injury. What made this happen? I personally believe the answer is simple "Pride." This was a joint effort of many people, who took pride in what was being accomplished. From the very beginning of the project to the conclusion, it was a team effort, from a consultant who was proud of its design and plans, to the contractor who took pride in quality work in a masterful way, to suppliers that responded to most every need, and to the proud employees of the Union Pacific Railroad that felt a sense of looking to the future.



THE BEHAVIOR OF RAILROAD AND HIGHWAY BRIDGES AFTER SEISMIC ACTIVITY IN MEXICO

By: Ing. Gonzalo Rivera Diaz* and Ing. Baltazar Campos de la Fuente**

The objective of this presentation is to report on the general condition of bridges of the highway and railroad networks in Mexico as a consequence of the most serious earthquakes the country has experienced in the last few decades. It is hoped that these results will contribute to improving the panorama in North America for this type of phenomenon.

Introduction

Right after earthquakes of September 1985, whose epicenter was located off the coast of Lazaro Cardenas in the Pacific Ocean and which registered a magnitud of 8 on the Richter scale, about 250 bridges of the highway and railroad network in the epicenter region of the states of Guerrero, Michoacán, and Colima were inspected. The purpose was to detect and evaluate the damage caused by these earthquakes in order to take the necessary emergency measures to guarantee safety of the users of the network.

Of the total of inspected structures, aproximately a third suffered minor damage, five presented moderate damage, and transit had to be suspended on only three highway bridges due to heavy damage.

In the area close to the epicenter, there is the Corondiro to the port of Lazaro Cardenas line with concrete bridges of big spans and height many of them designed on a curved layout.

Some of the highest metallic viaducts on the Mexican rail network are found on the Corondiro to Uruapan Railroad line. It is important to note that no railroad bridge suffered significant damage.

Seismic activity was registered again on October of 1995. Its epicenter was off the coast of Manzanillo. and reached 8 degree on the Richter scale. Same as with the 1985 earthquake, an inspection and damage evaluation was undertaken in the states of Colima and Jalisco where the line connecting Guadalajara to the Port of Manzanillo is located. In this track, there are important metallic bridges with riveted or articulated trusses.

Damage was found to be minimal in all cases.

Description of Most Common Damage Types

Following is a description of the type of damage most commonly observed:

Embankments and Track Deformations

In epicenter zones during the 1985 seism in Lázaro Cardenas, distortion was caused to embankments built with low shear strength materials and this resulted in the total deformation of the track. The embankment had to be rebuilt with stone materials.

Settlement on Bridge Embankment Approaches

Vertical drops between the deck surface of the bridges and the top of the access embankments were found inmediately after the earthquake. In some cases, these were up to 20 cms. and presented cracks along the lenght of the embankment.

This problem was observed in crossings built on soft soil, where bridges frequently shift on their deep foundations (piles or cylinders), while the embankments remain in their position. The relative sinking of the embankment with regard to the bridge usually takes place during a relatively long period of time, starting from the moment the structure enters into service.

Due to its dynamic compacting effect, the quake acts as accelerator of the settlement process in embankments built with insufficient shear strength material and also causes a settlement of the ground around the foundation.

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Transverse Displacement of the Superstructure

After the earthquake, relative transverse shifts between adjacent spans were frequently observed in bridges formed by several freely supported spans. These were manifested by disalignments of curbs and railings.

In general, these were small shifts of between 2 to 5 cms., and only two highway bridges presented serious shifts in the order of 20 cms. Several years ago, lateral support mechanisms were placed on the caps piers and abutments to prevent this transverse movement.

Lateral supports on some bridges were inefficient for this purpose since they cracked under the effect of the lateral force. Steel plate bearings and anchor bolts proved to be more efficient, since no damage was reported with this device. They are normally used in trusses and in metallic bridges in general.

Damage on Expansion Joints

Frequently, damage to expansion joints between consecutive spans of the superstructure were found, indicating the occurence of strong longitudinal movements during the quake. It was common to find these joints open, with no sealing material. The longitudinal movements caused continuous knocking between contiguous spans which, in turn, caused the concrete deterioration of the joint. In some cases the spans were moved from their original position and the joints were completely opened or closed up.

Damage on Bearings

The bearing mechanisms used by the superstructure to transmit loads to the substructure are critical to the behavior of the bridges during seismic activity.

In our country, the oldest bearing mechanisms are made of steel. These were substituted for concrete and lead bearings because of maintenance problems and their high cost. Lead pads have since been substituted for neoprene elastomeric bearing, due to poor results obtained with the sheet lead. It is worth while to mention that trusses still use metallic bearings. If properly maintained, they perform very well during seismic activity.

In one particular case, a serious failure was found in a steel bearing of a highway bridge due to cracking of the concrete cap of a pier and also at the base of the superstructure. It is probable that this failure was caused by the bearing's lack of flexibility due to the corrosion of its steel plates.

Some structures using lead pads showed vertical drops between consecutive spans after the sudden collapse of the bearing device. In this case, it seems that the earthquake accelerated the deformation of the lead.

In the former neoprene bearing mechanisms, the mobile support was made with several loose neoprene plates sandwiched between the steel sheets. Some of these bearings collapsed during the earthquake.

Modern flexible mechanisms were made by melting the neoprene plates in a mold containing the steel sheets to create an integral device. The behavior of this type of bearing during the quakes was an improvement over the others. Only in one case did the bearings tear and had to be replaced, and this was due to the violence of the movements.

These bearings were used in the railroad viaduct located in Km 35 of the Ajuno-Caltzontzin stretch. It is a box section continuous superstructure made of prestressed concrete and lodged in a sharp horizontal curve, with a central 80-meter span and two 60-meter laterals with 60-meter high piers.

It could be deduced that one of the superstructure ends took maximum transverse movements of 35 cms. After the earthquake, it remained with a permanent transverse displacement of 2 cms. These movements did not cause damage to the structure; they only broke the bearing. This bridge was under construction.

Damages on Retaining Walls

Several retaining walls were underdesigned to resist this type of phenomenon and, as a result, some walls cracked and others collapsed.

Damage on Substructures

As we know, the elements of the substructure are specially designed to resist the effects of horizontal loads over the superstructure.

Careful examination of inspected bridges revealed the following faults:

A masonry pier suffered vertical sinking of approximately 15 cms., and also presented a cracked joint and a generalized dislocation of the stones along its main body.

A pile bent of a concrete bridge presented a shear failure in its cap.

It is worthy to note that a 30-meter highway pier bridge hollow circular section did not collapse totally and could be repaired, despite the fact that erosion had seriously removed the concrete away at ground level, leaving most of the steel reinforcement bare and reducing the concrete section to 50% of its original size.

Damage to the substructure on a highway bridge at the access to the Island of Cayacal in the industrial port of Lazaro Cardenas, Michoacan is of special interest. This bridge is made up of two twin structures, each one built with six 30-meter simple spans.

Piers and bents placed on reinforced concrete cylinders constitute the substructure. Each pier is composed of a central column with a double cantilever cap.

The union between the column and the cap of each pier developed a serious failure. Besides fractures in the concrete, evidence of plastic deformation in the column's principal rods were observed.

Traffic had to be completely suspended on one of the bridge's twin structures, and low speed traffic on the other structure was stablished. False works were intalled until repairs were made.

The main reason for this damage was the proximity of the bridge to the earthquake's epicenter, which reached IX on the modified Mercalli scale. This intensity is greater than indicated by design norms.

Contributing causes could be the lack of ductility in the conection between the cap and the column due to insufficient lateral reinforcement, and to the effect of rotational inertia of the superstructure's mass that is applied to piers with long cantilevers caps.

Comments on Bridge Design in Mexico

Starting in the 1970's, highway bridges have been designed according to AASHTO specifications, applying the Equivalent Static Force Method. Since 1980, the Response Spectrum Method has been used for larger structures.

Railroad bridges built before 1970 were designed to withstand wind, lateral forces from equipment, centrifugal force (for curved layouts), longitudinal force of braking and traction, but not to withstand earthquakes. After 1970, an earthquake design for railroad bridges following AASHTO standards was introduced, and dynamic analysis was used for longer bridges.

The Mexican Republic was divided into four earthquake regions for the earthquake design. Additionally, data on speeds and ground acceleration for return periods of 50, 100 and 500 years were used.

Conclusions and Suggestions

It can be deduced from the damages to the bridges that have been described, that the effects of the October 1995 and September 1985 earthquakes were moderate, especially if they are compared to their effects on buildings in Lazaro Cardenas, Michoacan, Ciudad Guzman, Jalisco and Mexico, City in 1985, as well as to the damages in Manzanillo, Colima in 1995.

Explanations of Seismic Resistance of Bridges

- Actually, bridges are not very tall structures; they are made of massive elements of great stiffness. Therefore, their fundamental oscillation periods are short, less than 0.5 seconds. Consequently, their response to movements like those caused by the earthquakes of September 1985 and October 1995 is less than of flexible structures with greater fundamental periods.
- As a defense against damage during the moment of the earthquake's maximum intensity, the foundations of bridges are generally built at great depth, into the strong layers of soil.
- Expansion joint and bearing mechanisms are isolating elements and contribute significantly to the dissipation of energy, thereby reducing the forces transmited to the substructure.
- Continuous rails on the bridges constitute an element of resistence to horizontal movements provoked by earthquakes.
- The magnitude of live loads on railroad bridges provides vigorous structural elements that respond very favorably to seismic activity.
- The design of the substructure's components enclose other horizontal forces, like wind, lateral forces from equipment and longitudinal forces, which very often produce horizontal loads bigger than those caused by an earthquake.

Study Measures

- The damage to bridges might be more significant in other seismic events with different characteristics of intensity, or given the case that structures might be constructed with more flexibility.
- For this reason, further analysis should be conducted for important structures for which might be considered a joint work between the ground and the foundation elements for the different seismic solicitations.
- The dynamic behavior of the components used for bearing mechanisms and expansion joints should be deeply investigated. The main problem in current bridge analysis is the correct modeling of these elements. Further investigation will allow us to develop new types of bearings, like the ones that have been recently introduced, where the combination of rubber and lead has produced integrated bearings that absorb shocks better through friction and, therefore, improve the seismic response of the structure.

Recommendations

- The Equivalent Static Force Method, usually applied in seismic design, offers satisfactory results for rigid frame structures and a conservative approach for simply supported spans, since the isolating effects of the bearing mechanisms of the latter are generally not considered.
- Restraining features or earthquake stoppers, are usually underdesigned, since lateral load is evaluated by considering an implicit ductility factor that the lateral support is incapable of delivering. Therefore, it is recommended they be designed with a ductility of Q=1. It is also recommended that the space between the superstructure and the stoppers be filled with a neoprene lateral pad to improve performance.
- The displacement that currently is anticipated, is much more inferior to those which can be presented under an intensive earthquake, consequently it is accurate to improve the quality of the sealing materials of the joints to increase its durability and its stability. In important bridges, it is convenient to restrict the longitudinal movement of the super structure, installing prestressing bars in the joint of the two adjacent spans. These will allow temperature movement, but not seismic. This caution is particulary important for cantilever strctures such as Gerber type beams.

- The access embankments made of soft soil should have wider bases, obtained by means of benches or by more extended slopes, to prevent the settlement of the foundation by intensive earthquake effects or by consolidation on the long term.
- The damages observed in the masonry elements are attributed to the poor quality of work. Therefore, for more important bridges in seismic zones, such materials must be avoided in favor of the reinforced concrete for substructures.

POSITIVE TRAIN SEPARATION

By: Larry R. Milhon*

Introduction

The Burlington Northern Santa Fe and the Union Pacific initiated a joint pilot project to develop a derailment prevention system called Positive Train Separation. As might be obvious from its name, this system is being put into place to eliminate the most serious of train derailments, train collisions.

GE-Harris has been contracted to build a safety overlay system that will prevent trains from colliding. This means that the system does not replace any of the movement authority controls in place today. Train movement will still be authorized by signals, track warrants, and other safety systems currently in use. The PTS system merely enforces these rules.

The pilot project region covers roughly 863 miles of BNSF and UP track in Washington and Oregon. Several sections of this territory are covered by joint trackage agreements between the two railroads. The selected area allows the project to test all of the fundamental types of operations, such as CTC, ABS, non-signaled, and non-controlled. This region was also selected to address some of the issues of interoperability between railroads.

The system will be implemented in a series of four releases starting in August of 1996. Each release will add increasing levels of detail and each will build on the previous release's capabilities. The project completion is currently set for December 1997.

Why PTS

Railroads are always working on derailment prevention, either individually or collectively through the AAR. Improvements in rail equipment, track structures, and railroad operations have all contributed to the almost continuous reduction in derailment rates.

Even though railroads have made steady improvement in overall derailment performance, train collisions continue to cause property damage and more importantly, loss of life. PTS is designed to move toward the elimination of these derailments.

PTS Technical Issues

The PTS system is logically composed of three functional elements, the server segment, the communications segment, and the locomotive segment.

The PTS server segment will be located in the central office for each of the railroads and will provide connections to both the Computer Aided Dispatching systems and the main railroad Information Systems computers.

The server will observe all train movement authorities granted by the CAD system and by the dispatcher and transmit these to the computer onboard the PTS equipped locomotives. Prior to sending the PTS enforceable authorities, called PTSEA's (pronounced PIZZA), to the train the server checks to make sure they do not conflict with any of the rules it has been charged to monitor. Without a PTSEA, a PTS train cannot be moved.

The server communicates with the locomotive using a data radio - the communications segment. This is the same communications system that is being installed for codeline replacement and other data communications functions. The radio coverage area will have to be increased in some areas to accommodate the added requirements. For the pilot, each railroad will equip ten locomotives. Each PTS locomotive requires the installation of an onboard computer, a location determination system and, if not already present, a data radio.

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The onboard computer (OBC) receives the train's movement authority, PTSEA, from the PTS server and, using onboard location information, watches the progress of the train. As the train nears the end of its authority it will most likely get an additional authority and again the onboard computer does nothing but watch.

If on the other hand the train does not receive an additional authority and the train nears the end of its current authority, the computer will issue a message to the engineer warning of the situation. If the engineer does not make sufficient progress to stop the train inside its authority limits the onboard computer will cause a penalty brake application and will stop the train inside its authority limits. Hence, positive train separation.

To accomplish this, the onboard computer is constantly computing the trains "safe braking distance." This is the distance the train requires to stop using a full service brake application from its current position on the track. This calculation takes into account weight and length of the train, the train's current speed, the track under and in front of the train, and any other information that might be relevant.

Location Determination System

A very important piece of the PTS system is the Location Determination System (LDS). In order for the PTS system to know precisely when to apply its protective measures, the system must know accurately where the train is at any given time. When a train passes a turnout, the system must be capable of determining which track was taken regardless of the type of operation. In CTC territories, the CAD system could provide information regarding which track was occupied. But, in ABS and dark territories some other system is required.

The LDS is composed of several pieces, a Differential Global Position System (DGPS) receiver, a gyro, an axle generator and a track database. These pieces are used together to verify with great confidence where the train is and which track it's on. This allows the PTS system to be able to stop the train within its authority limits and to make sure the train is operating within its current speed authority.

A natural question—why not GPS alone? Due to a variety of terrain problems, GPS alone does not appear to have sufficient coverage to handle all situations. Mountains and heavily wooded areas sometimes block the view of enough satellites making it difficult to get accurate information. Tunnels create another difficulty. But a combination of the above systems should be adequate to provide the detail needed.

During the pilot project, track transponder tags will be used to precisely identify the train's location. This will allow the GPS based LDS to be tuned and validated in a variety of conditions. To accomplish this, all major decision points will be bracketed with tags to precisely identify where the train is and which track it's on. The final PTS system will not require track tags for correct operation.

Interoperability

One subject of major importance is interoperability. What does it mean? What does it encompass?

Both BNSF and UP have been participating in AAR sponsored discussions addressing issues of interoperability. It is an issue that will be key to the success of this type of system. One of the fundamental reason the two railroads chose to do a combined pilot was to address the concerns of joint operations.

For the pilot, each railroad would like to operate trains on joint trackage without conflict and pass trains off to each other at track speed.

There were several interoperability issues that were resolved through negotiation. But, one stands out as more of a technology issue. Historically, the BN has used an ARES data radio protocol whereas the UP uses an ATCS standard. To make things even more interesting the BN radio operates in the VHF band while the UP radios are in the UHF frequencies. These issues are being addressed in the pilot region.

The Schedule

The pilot project being developed will be implemented in a set of four releases. The first release will start with basic system startup tests and subsequent releases will build upon previous releases.

In Release 1, currently scheduled for August 1996, two locomotives on each railroad, will be equipped with all of the basic hardware, the onboard computer, the data radio, the LDS, the crew interface, etc. This initial release testing will be performed on the Harbor Line Subdivision for the BNSF and on the Kenton Line Subdivision for the UP.

Startup and shutdown of all the system will be tested. Some of the required data will be supplied manually in Release 1. The location determination system will primarily be based on track tags and the wheel tach. Also in Release 1, a predefined track warrant will be downloaded to the locomotive and verified by the crew.

Release 2, early 1997, adds several new levels. The track data will now be uploaded from the server to the locomotive for the onboard calculations. System logging of significant events should be in place on the server.

Full locomotive display capabilities are planned for Release 2. Also, DGPS will be added to the location determination system in this release. The CAD interface is also scheduled to be in place at this time. The BNSF CAD interface may be slightly delayed due to merger changes, so certain of those functions may be simulated for Release 2.

By summer 1997, Release 3 will start to add significant capabilities to the PTS test. Both the CAD and the MIS interfaces should be in place and functional. High speed data transfer for database updating will be tested along with ad-hoc database queries.

Full LDS based capabilities should be ready and speed enforcement should be enabled. This release initiates the beginning of multiple train testing with the verification of a close following move. CAD movement authority processing will be tested for non-ABS, ABS, and CTC track.

Release 4 provides closure for all system testing. All systems startup, initialization, and termination will be verified. System redundancy will also be tested. Joint operation server to server functions will be tested. Full track database capabilities will be tested.

The key step in Release 4 is the BNSF and UP interoperability testing. If all goes as planned Release 4 testing will begin in October 1997 with completion December 1997.

The Future

Where do we go from here? Looking toward the future, all railroads are thinking of ways to increase capacity without the tremendous cost of adding infrastructure. The BNSF and the UP are looking for ways of better utilizing current investments, including PTS, to accomplish these goals.

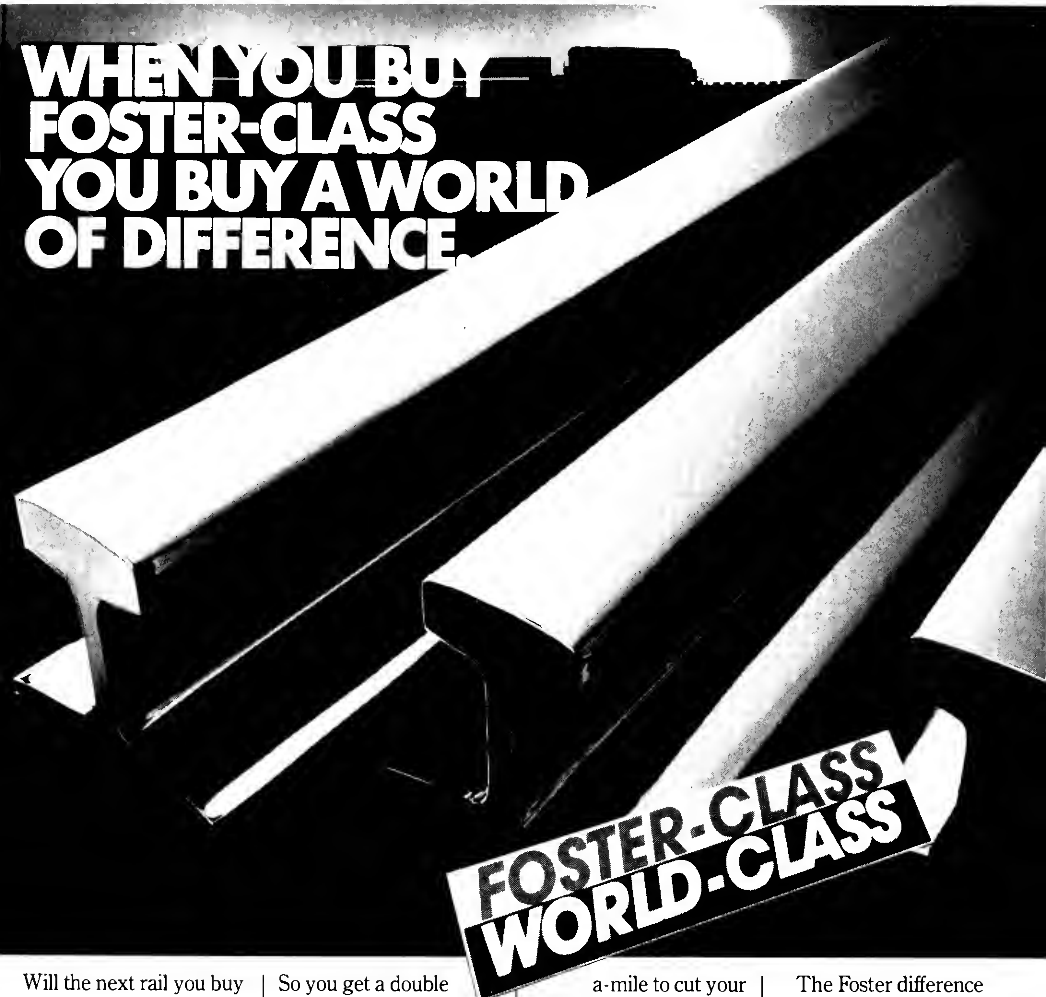
It seems natural that one might add more sophisticated planning capabilities to the architecture being built for PTS. The improved location knowledge available from PTS could provide for more precise dispatching of trains and therefore create added capacity through improved meets and passes.

PUBLISHED AS INFORMATION
COMMITTEE 4—RAIL

Chairman: S. G. Atkinson, Jr.

Consolidated Report of Rail Shipped to North American Railroads from North American and Non-North American Producing Mills in1994 by Weight and Section

Weight	Section	N. American Tons Shipped	Non-N. American Tons Shipped	Total	% Total
100	RA	7,741	1,000	8,741	1.23
100	RE	2,522	0	2,522	0.36
115	RE	90,682	27,871	118,553	16.73
119	RE	772	0	772	0.11
122	CB	0	900	900	0.13
132	RE	19,445	9,258	28,703	4.05
133	RE	47,947	36,165	84,112	11.87
136	RE	346,001	115,969	461,970	65.22
140	RE	0	0	0	0.00
Other	Includes Girder Rail	2,019	83	2,102	0.30
TOTAL		517,129	191,246	708,375	



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COMMITTEE 14—YARDS & TERMINALS

Chairman: Thomas B. Schmidt

Assignment D-1-93: Design of Municipal Solid Waste Railroad Terminals

Committee 14 is proposing additions to the Manual relating to the design of municipal solid waste railroad terminals based upon the information given below, but written in Manual format. Comments, which are solicited, should be addressed to AREA headquarters.

Summary

As many municipal solid waste (MSW) land fills near cities are reaching capacity, more metropolitan areas are turning to railroad hauling of MSW to landfills in remote locations to solve their disposal problems. Undertaking this method of handling MSW involves several design considerations, such as: the locations, ownerships, and operators of the waste loading, unloading and landfill facilities and equipment; the types and ownerships of the railroad cars, containers, and truck trailers to be used; the lengths of truck and train hauls; the sizes of waste consists or unit trains; and the frequencies of train operations. Facilities should be laid out to enable the efficient movements of rail cars and trucks with a minimum of conflicts between them. Constructing MSW handling facilities often involves extensive governmental permitting and much public opposition, which can result in long delays before the start of the construction processes. These facilities should be constructed so as to shield the public from offensive sights, lights, sounds, odors, dust and vermin; truck traffic on city streets should be minimized. Provisions should be made in the designs of all facilities for future expansions.

Introduction

Increasing amounts of municipal solid waste (MSW) are being moved by rail to disposal sites. The tonnage of waste being moved by train is also increasing. With landfill sites becoming more scarce near the points of high waste generation, MSW must often be transported considerable distances to suitable disposal sites. Most disposal sites are sanitary landfills, but a few are electric power generating plants which burn waste for fuel, so-called "waste to energy" plants. With the latter, methods of hauling away and disposal of the resulting ash must be provided.

Waste may be moved in railroad cars which are constructed or modified for MSW transport, in truck trailers carried on specialized flatcars (TOFC), in containers carried on specialized flatcars (COFC), or in convertible highway-railroad truck trailers (such as Road-Railers), all of which must be watertight to eliminate spillage enroute. Old box cars used for waste hauling are often modified by replacing the roofs with removable covers, and adding interior dividers. The selection of the type of vehicle to be used will be dictated by the anticipated amount of waste to be moved, the configuration (existing or proposed) of the collection system, the proximity of the disposal site to the rail terminal, and the proposed ownerships of the various facilities and equipment. A complete systems analysis and economic justification of a proposed MSW operation, from collection to disposal, is recommended before the types of waste conveyance are selected.

Site Selections

Site selections for municipal solid waste transfer stations and disposal landfills are sensitive issues which are subject to close public scrutiny and objections. In general, a site should have good highway access, have adequate space for the receiving, storing, sorting and handling of solid wastes, and accommodate the efficient transfer of those wastes between the road and rail modes. The sites should be compatible with the surrounding land uses, and be properly zoned for the activities contemplated. Local zoning restrictions may eliminate many otherwise practical site locations.

The construction of solid waste handling facilities is heavily regulated in most public subdivisions under their powers to protect the public health and environment. Designs should reflect compliance with all permitting laws, regulations and ordinances. Cleanliness, odor control, and vector (vermin) control are three of the most important requirements for waste handling facilities. The operation of these facilities should also be efficient and economical.

In general, siting in residential areas is less preferred than in rural or industrial areas. For a terminal proposed to be near residences, extensive visual and noise shielding, in addition to odor and vector control, may be required as conditions of operating there.

Any locations proposed for solid waste terminals should be thoroughly investigated for environmental regulatory issues, such as: wetlands; threatened or endangered species; archaeological sites; and flood plain proximities. Before proceeding with the planning for any site, the permitting issues involved should be well understood, and plans formulated to address each of them. The permitting process and possible accompanying litigation may add several months to a proposed construction schedule, and may, in a worst case, destroy a project altogether.

After prospective sites have been screened for environmental and other regulatory requirements, the site which best serves the logistical needs of a solid waste transport system is generally the best location. If possible, room should be provided for the orderly expansion of facilities to accommodate future increases in the volumes of wastes to be handled.

The Transfer Station—Truck To Rail

The originating rail terminal, called a transfer station, should be located so as to balance the collection truck hauls to the terminal from all of the waste generating areas to be served. A good terminal location will minimize non-productive times for trucks, and will promote the efficient flow of

Figure 1.
Rabanco Corporation MSW Train on SP's Cascade Line near Dunsmuir, CA (photo courtesy of Bob Morris Photography, Dunsmuir, CA).

waste through the terminal and the timely movement of trains to and from the terminal. Minimization of interchange moves and/or crew changes, and the use of run-thru locomotives are also desirable from the railroad operations standpoint.

The transfer station site should have easy access to a trunk highway system, and not rely on the use of city streets. Considerations should be given to truck axle loadings, turning radii, and the dense traffic generated by solid waste transport operations. The establishment of the locations of entrance and exit points at the terminal should be coordinated with local highway agencies. As waste disposal charges are usually based upon the weights handled, weighing of the loaded trucks when entering the station may be required, as well as when leaving empty if the truck tare weights are not known. Weighing of loaded rail cars may also be necessary for railroad billing calculation purposes unless agreed-to average lading weights are used.

If classification of waste materials to reduce volumes is to be performed prior to loading them into rail cars or other containers, additional space will be required to accommodate the sorting out of compostable and/or recyclable materials. If recyclables are to be involved, then the facilities to store, load, and ship them out of the station by truck and/or rail should be provided.

The transfer station should be designed to accommodate the loading method chosen to transfer the waste from inbound trucks to outbound rail cars. Where solid waste is to be delivered directly to the rail cars, a hopper which receives the truck hauled waste, and which discharges directly into the rail cars may be used. Another means is for the waste to be dumped by the trucks directly onto an area provided on the station floor, after which it may be mechanically shredded and/or compacted, sometimes into bales. Then front end loaders or other mobile equipment are used to pick up the waste and load it into the rail cars. Hydraulic cranes equipped with grapples (a.k.a. knucklebooms) may be used to assist in loading and compacting the waste in the cars by rearranging and spreading it, while also removing rejectable materials, such as gas canisters, chemical drums, paint and oil cans, etc.

Covers are used on open-top rail cars, containers and truck trailers to contain the waste during transit. These covers may be made of metal, canvas, plastic, or netting. Provisions should be made to remove these covers efficiently for the loading and unloading of wastes, and to replace them when those operations are completed.

For trailer and container loading on specialized flat cars, refer to the AREA Manual, Chapter 14, Part 4.2, Design of Intermodal Terminals. With these types of operations, the transfer station may be some distance from the rail loading point, which may be at a new or existing railroad intermodal transfer facility.

Unloading (Disposal) Facilities

The destination rail terminal should be as close as possible to the ultimate disposal site to minimize the truck cycle time between that terminal and the actual disposal site. If train hauled truck trailers or containers are used to transport the solid waste, the train unloading operations are similar to the ones used for other rail container or trailer freight in intermodal terminals. (See AREA Manual, Chapter 14, Part 4.2, for additional guidance in the design of these terminals.) Modified box, gondola, and hopper cars carrying waste may be unloaded using rotary dumpers (either single-car, or multiple-car with rotary couplers), clamshells, or backhoes. (Many rotary dump installations are similar to those described for random, uncoupled car dumping in the AREA Manual, Chapter 14, Part 4.4, covering the design of Bulk Solid terminals.) As more unit trains are dedicated to waste haul, their terminals may come to closely resemble those of unit bulk solid train terminals for rotary-dump cars. A system should be included in the terminal design for removing any car or trailer covers that are being used and returning them to the originating transfer station.

The waste is transported from the rail unloading facility to the landfill site in the arriving trailers or containers on chassis, or in large truck trailers loaded at the rail terminal by hoppers, conveyors, backhoes, or end loaders. The chassis of these trailers or under the containers may be equipped to elevate for end unloading, or they may be unloaded using a tipping device. Final placement of the waste may be accomplished using grader-scrapers, bulldozers, front end loaders, patrol graders, and/or earth compactors.

The pits at the disposal landfill are usually lined with rubber or plastic membranes, which are sealed and underlain with impervious clay layers to prevent the escape of liquids. They also may have a system of perforated pipes to collect any leachate from the landfill, and to convey it to holding tanks for ultimate treatment and disposal. If this leachate is to be conveyed to another site for treatment and disposal, loading and unloading facilities for tank trucks or rail tank cars must be provided. Collection pipe systems are also usually placed in landfills to gather and dispose of flamable gases (usually methane) generated by decomposition of the wastes.

Aspects Common to Both Transfer and Unloading Facilities

Sufficient utilities should be available at both waste loading and unloading sites. Electric power will be needed for lighting, and for waste conveying, processing, shredding, and/or classifying equipment, for hydaulic machinery, for air compressors, for maintenance operations, and for wastewater pumping and treatment. Water for fire suppression is of prime importance, and is also needed for sanitary and washdown purposes. Sufficient sanitary sewers and wastewwater treatment facilities must be constructed onsite or arranged for offsite to handle wastewater drainage and car/trailer/container cleaning effluents. If stormwater runoff is liable to be contaminated with wastes, it may also have to be treated before it is released. Compressed air may be needed for cleaning operations and for charging train air brakes.

Care must be taken to frequently clean up any spillage of wastes occuring during the (un)loading operations. The optimum operation moves wastes from their sources, through the transfer station, on the train haul, and through the unloading facility and into the landfill as rapidly as practicable. Delays encourage the attack of the wastes by vermin and the generation of odors. Most sanitary landfill operators cover incoming wastes with earth the same day that they are received.

Layout of Trackage

In general, the trackage at a waste handling facility should be extensive enough to easily handle the longest unit trains or cuts of cars expected with simple and direct moves, with a minimum of switching, and with as few conflicts as possible with other rail movements, and with truck and other equipment traffic in those facilities.

Track curvatures and turnout configurations should support train and switching operations compatible with the type of railroad waste hauling equipment to be used. Loading, unloading and holding tracks should be as straight and as level as practicable. Vertical curves between grade changes should be at least 100 ft. (35 M) long. Where mainline hauling locomotives are to operate, the minimum horizontal curve radii should be at least 450 ft. (140 M), with at least 100 ft. (35 M) of tangent between reversing curves; turnouts should be No. 8 or flatter.

It is recommended that continuous welded rail (115# or heavier) be used in the paved areas of the facilities. While jointed rail performs satidfactorily in open trackage, joints can cause heaving and pumping problems in pavement. Grade crossings which are to be used by heavy waste hauling trucks and intermodal lifting equipment must have strong pavement sections or crossing modules underlain by substantially constructed subgrades.

The unit train receiving track at a facility, or the lead track thereto, should be long enough to hold clear of the main track the normal length of unit train to be run, including locomotives. The cars being received may be stored on the receiving track while awaiting the next shift to handle waste at that facility, which is the method of operation often preferred by the waste handlers. If the receiving track does not have access to a main track on both ends (i.e.—is not double-ended or a balloon track), then an escape track may be needed for locomotives to run around the cars which they deliver. If road-haul locomotives are to stay in a facility, an environmentally safe holding track should be provided where they can be stored and also receive fueling, servicing, and light repairs.

Tracks used for (un)loading waste should be level, if possible. If not, gradients of no more 0.1% (downgrade in the direction of car (un)loading movements, if any) should be used. Modified box, hopper, and gondola cars may be moved through an (un)loading facility by using a car indexer, barney, rabbit system, cable and winch, mobile car-mover, small industrial locomotive, or a switching or road locomotive which has been modified to be able to operate continuously at very slow speeds.

At facilities utilizing TOFC and/or COFC technologies for waste transport, the trailers and/or containers are lifted onto and off of their specialized flat cars with the commonly used intermodal transfer equipment, such as straddle cranes, gantry cranes, frontend lifters, etc.. With this method the road-haul locomotives usually move the unit trains directly to and from the (un)loading tracks. The intermodal flat cars, often articulated, are usually left in place between arrival and departure, unless some of the cars require heavy repairs. Light repairs, including changing out wheel sets, are usually performed by using mobile equipment where the cars sit on the (un)loading tracks. Independent tracks should be provided for holding new and bad order wheel sets. If there is room at a facility, a separate spur for heavy repairs may be provided.

To save time for the road-haul locomotives moving cars out of a facility, whole unit trains or shorter cuts of cars that are ready to leave should have their air brakes charged prior to the arrival of those locomotives at the facility. This charging can be accomplished by using either a portable air compressor or a stationary air compressor with a pipe distribution system.

Buildings

The largest building at either the originating or terminating facility is the one where the transfer of wastes between modes of transport takes place. This building will be even larger if dumping by the public is allowed, and sorting and/or salvaging of materials take place within it. These transfer operations are done inside to provide: a dry workplace out of the sunlight and weather, to contain and capture dust, noise , and odors; to control vermin; and to shield the operations from public view. Where locomotives and rail cars operate through and beside buildings, proper overhead and side clearances must be provided. (See AREA Manual, Chapter 28, Clearances.)

Other buildings that may be required are: repair garages for trucks, trailers, grading and other equipment; an office building; a truck scale building (may be part of office building); locker and washroom facilities for dump and railroad employees; gate (security) houses; a building to house railroad car repair, locomotive supplies, and track materials; a trailer tipper building at the landfill. A railroad scale may be needed if car weights are required as part of the plant operations, for overload detection, or for haulage billing purposes. Preferrably this should be a remotely-read, weigh-in-motion scale.

Security

The main concerns involving security at these facilities are about the dangers of injuries to, the unauthorized dumping by, and attempted salvaging by trespassers. The entire (un)loading facility should be fenced, with all gates being locked when not in use. Fencing around areas visible to the public may require slats to inhibit viewing. Where a facility produces load noises, sound walls may be required. An adequate level of lighting should be provided during all hours of darkness to discourage trespassing. Closed circuit TV monitors may be needed where the number of security guards is small. Adequate signing should be provided to inhibit trespassing and to promote the orderly flow of authorized traffic.

Vector Control

Municipal wastes contain animal and vegetable matters that attract and propagate many species of vermin and other animals. Flys, maggots, yellowjackets, fleas, birds, cockroaches, mice, rats, squirrels, opposums, raccoons, skunks, cats, dogs, and even deer and bears may cause problems. The best control method is to move the wastes received as quickly as possible from origin to disposal, either into a sanitary landfill that is spread with earth freqently or daily burned in a power generating plant. Housekeeping of the areas where wastes are handled, sorted and stored is of the utmost importance. Thorough daily cleaning, often with high-pressure water and/or air, of the areas and equipment involved is useful in housekeeping effort. Some vector control may be obtained through the judicious uses of poisons, sprays, desiccants, traps, crys and/or decoys of natural predators, revolving lights, and fences, and enclosed buildings. Any lengthly delays in the handling of wastes will also produce pronounce odor problems. Keeping wastes covered, out of the sunlight and away from heat will help delay the onset of putrefaction.

Some Current Examples of Hauling Municipal Solid Waste by Rail

1. Bergen County, New Jersey, to Charles City, Virginia

Municipal Solid Waste (MSW) is brought by local collection trucks to the Bergen County Utilities Authory's transfer station, which has been operated since 1991 by Chambers Waste Systems of New Jersey. The MSW is dumped on the transfer station building floor, and the density of the waste is increased using a landfill compactor. A front end loader is used to place and compact the MSW into a 8' x 10' x 20' (2.4 m x 3.0 m x 6.1 m) patented, open-top, aluminum container. A loader places a lid on the container, and then lifts it onto an intermodal trailer chassis. It is weighed, and set in a staging area outside of the transfer building to await loading. These containers a loaded, four to each intermoadal car, along a 450 ft. (140 m) double-ended track which also passes through the transfer building where baled MSW was formerly loaded into box cars. Chambers uses a GP-9 to switch cars on the 1.5 miles (2.4 km.) of track available at the tranfer station.

The 325 mile (520 km.) haul of the 600+/- tons (540+/- tonnes) of MSW generated daily is handled in regular freght train service by two railroads: Conrail from Bergen County, NJ to Alexandria, VA; and CSX from there to the Chamber's Charles City County, VA, landfill. Each MSW consist is yarded at three different locations enroute.

At Charles City County, three spurs, having a total capacity of 23 intermodal cars, have been constructed next to CSX's Richmond to Newport News line. Here a container is tranferred to an intermodal trailer chassis using a fork lift., and is then trucked to the sanitary landfill six miles (9.7 km.) away. It is weighed for a second time, and is then moved to the working face of the landfill. A specialized forklift is used to remove and hold the lid from the container, and then rotate the container to a 55 degree angle to unload the MSW from it in less than one minute. The container is then covered, replaced on the chassis, and retuned to the rail spurs for loading for the rail trip back to New Jersey.

2. Seattle, Washington to Columbia Ridge, Oregon

In 1991 the Union Pacific began hauling MSW from Seattle to the Washington Waste System's Columbia Ridge Landfill in eastern Oregon. This non-recyclable waste, about 600,000 tons (540,000 tonnes) per year, is collected by trucks throughout the Seattle area and brought to one of four transfer stations, all of them remote from the railroad. There the waste is compacted into 37 ft. (11.3 m.) "slugs", each weighing up to 32 tons (29 tonnes), which are then end-loaded into especially built, leak-proof, 40 ft. (12.2 m.) long containers. Then these containers each have their open end closed, are loaded onto chassis, which are then trucked to the UP's Seattle intermodal facility. There are a truck entrance and a gate house dedicated soley to MSW handling in order to insure a 20-minute turnaround time through that facility. The containers are loaded onto UP supplied, 80 ton (74 tonne) capacity double-stack cars, two per car, by a railroad packer used only in this MSW service.

Three times a week, unit trains each carrying 100 to 120 containers of MSW leave Seattle for the 330 mile (530 km.), 20 hour long trip to the landfill in Oregon. There the containers are taken from the trains with lifting equipment at a new unloading facility, placed on chassis for the short haul to the dumpsite, where they are unloaded using end-tippers. The empty containers are then placed back on the unit trains for return to Seattle.

3. Roanoke, Virginia, to Smith Gap, Virginia

The Roanoke Valley Resource Agency (RVRA), working with the Norfolk Southern Corporation, developed an inovative waste-by-rail system. Starting in 1994, RVRA began moving about 700 tons (635 tonnes) of MSW per day 33 miles (53 km.) to a new 1,200 acre (480 ha.) landfill site near Smith Gap in the Blue Ridge Mountains. This MSW is generated by the City of Roanoke, the County of Roanoke, and the Town of Vinton.

At Tinker Creek in Roanoke, a new 33,000 sq.ft. (3,065 sq.m.) transfer station, designed to resemble a historic railroad shop building, was constructed on 22 acres (9 ha.) of industrial land formerly occupied by an abandoned steel mill. There are three tracks inside this new facility.

Collection trucks unload MSW onto the concrete floor of the transfer station for inspection. The waste is then pushed by front end loaders over the edge of the floor into one of two hoppers, which, in turn, discharge into waiting, especially designed, open-top rail cars. These rail cars, each holding from 65 to 70 tons (59 to 63 tonnes) of MSW, are then covered with water-tight, lock-down lids by an especially designed crane, which had previously removed and held these lids during the loading process.

Each weekday night, a ten-car train, called the "Waste Line Express", moves the loaded cars 33 miles (53 km.) to a new landfill near Smith Gap, VA. The last 5.5 miles (8.9 km.) of this trip is over a new branch line which was constructed by NS soley to reach this previously totally inaccessible landfill site. Upon arrival, the locomotive positions the cars for unloading the following day. It then gathers the cars which were made empty previously and returns them to the transfer station in Roanoke. It also brings back any railroad tank cars fill with leachate recovered from the heavily-lined landfill for disposal into the City's industrial sewer connection at the transfer station.

The following day, a trackmobile positions the MSW cars, one at a time, under another overhead crane which removes, and then holds or stacks the lids from the cars for replacement after the cars are unloaded. A car is them moved onto a rotary car-dumper inside of a large tipper building, where the contents of the car are unloaded onto a concrete pad. The MSW is then loaded into 40 ton (36 tonne) capacity, off-road trucks for movement to the face of the sanitary landfill nearby.

Norfolk Southern has a 25 year contract with the RVRA to operate the "Waste Line Express". What is so unusual about this operation is the short, 33 mile (53 km.) rail haul. In the waste handling industry, a minimum rail haul of 150 miles (240 km.) is normally considered essential to return a profit.

4. American Canyon, California, to Roosevelt, Washington

In June 1995, the new Devlin Road Transfer Station in American Canyon (Napa County), California, was opened to load containers with MSW for the 710 mile (1143 km.) train haul to the Roosevelt Regional Landfill in Klickitat County, Washington. This transfer station is ownerd by the South Napa Waste Management Authority, which is a California joint power made up of the County of Napa, the City of Napa, and the City of Vallejo (in neighboring Solano County).The station is operated under a renewable 10-year contract by Rabanco Corporation of Bellevue, Washington., under the name of the Regional Disposal Co.. The sanitary landfill at Roosevelt, Washington, is also operated by Rabanco, where it receives rail-hauled MSW from several origins in Oregon and Washington.

Collection trucks bring municipal wastes from several locations to the transfer station, where it is dumped onto the station's concrete floor, and then is mechanically loaded into containers from one end. The leak-proof containers being used are 45 ft. (14 m.) long, and each can hold up to 28 tons (25 tonnes). Adjacent to the transfer station building, they are loaded onto double—stack rail cars, two per car, using sideloaders with overhead connectors.

A loaded waste-hauling train, consisting of 80 containers double-stacked on 40 intermodal cars, departs from American Canyon every four days (see Figure 1). These trains travel for 10 miles (16 km.)

on the regional California Northern Railroad from American Canyon to Suisun,CA; then 380 (612 km.) to Klamath Falls, OR, on the Southern Pacific; and finally 320 miles (515 km.) to Roosevelt, WA, on the Burlington Northern—Santa Fe.. The locomotives for these unit trains, which run through on all three railroads, are furnished by both SP and BN-SF.

At the Roosevelt Regional Landfill, the containers are lifted off of the rail cars and onto chassis for the short truck haul to the dump site, where they are unloaded using an end-tipper. The annual haul of MSW should equal 200,000 tons (180,000 tonnes). As yet, there are no return shipments of commodities via these unit trains. An interesting feature of the overall unit charge for the operation (currently being $60/ton = $66/tonne) is that it includes a special fee of $7 to pay for the closing of the former landfill in American Canyon, which was one of the oldest in California.

PUBLISHED AS INFORMATION
AMERICAN RAILWAY ENGINEERING ASSOCIATION
COMMITTEE 16—ECONOMICS OF RAILWAY ENGINEERING AND OPERATIONS

Chairman: Johnny M. Johnson

Assignment D-3-92: Economics of Train Delay Progress Report

By: David H. Noble*

Introduction

Train delays and their costs should be evaluated in every economic study of railroad operations. Delays create an irreplaceable loss of assets and earning power, and increase operating expenses. In many cases, a reduction or elimination of these delays will require capital expenditures. This cost should be balanced against the costs created by the delays. The purpose of this report is to define the causes of delays, to discuss potential costs, and to suggest possible solutions.

Unfortunately, the term "train delay" is imprecise, and consequently quantifying specific costs becomes difficult to impossible from a practical standpoint. Assignment of specific costs to a particular delay is difficult because of the ripple effect it may have on systemwide operations. A locomotive delayed at one point can delay a second train when it does not arrive in time for its normal assignment, and a through train delayed on one division may create delays several divisions later when normal meets with other trains are disrupted.

In the final analysis, the importance of train delay prevention depends upon management's commitment to customer service and to resources available to minimize service interruptions.

Determining Frequency of Delays

The first step in delay prevention is identification of actual delays. Nearly every railroad has a "Morning Report" of train operations for the previous 24 hours. These usually detail every delay to principal system trains and they offer a first step in isolating trouble areas or causes. More detailed studies can be made using the dispatchers train sheets, or even by posting observers in the dispatchers' offices. To be sure that any seasonal variations are included, the period studied should be as long as possible; however, if changes in operations have created the delay problem, previous records may be of little value.

Car movement records are also a source of delay information. In the final analysis, it is the timely movement of the actual car from the loading point to the final customer that is important. Since train delays will directly impact car transit time, both train and car records can be used to develop a picture of system delays.

Causes of Delays

Scheduling: Scheduling can result in bunching of trains in specific locations creating congestion. They can also result in trains meeting or passing where adequate sidings are few or far apart. Mixing passenger trains, preference freight trains, and drag freights can contribute to delays as faster trains attempt to pass slower trains.

*Member, Committee 16.

Congestion: Delays can result if traffic exceeds line capacity. This can occur seasonally or it can be caused by rapid traffic growth. Congestion will be most obvious where there is a restrictive segment of an otherwise high capacity route. Single track tunnels or bridges on double track lines, heavy grades, or a yard with limited track space are examples.

Power or Crew Shortages: Any delay will indirectly contribute to locomotive or crew shortages at the destination. If this is a continuing problem, it may indicate that the shortage is a cause of delays, not a result.

Frequently operations created delays can be eliminated by rescheduling. If traffic that is not time sensitive can be moved over the restrictive area when it will not conflict with other traffic, overall performance will improve. If this is not acceptable, line improvements will be needed. This can be as simple as lengthening critical passing tracks or as extensive as double tracking, line relocation, or centralized traffic control. Yard congestion can also be reduced by spreading train arrivals and departures more evenly, by changing train blocking procedures more evenly, by changing train blocking procedures, or by use of unit trains that require no classification at intermediate terminals.

Maintenance Oriented

Maintenance delays occur when planned work on track, signals or structures results in slow orders or otherwise restricts train movements. The potential for delays, and their costs, should be included during preliminary planning so that the most economical work schedule can be selected.

Failure or Accident Related

Equipment Failure: No mechanical device will operate without unexpected failure, and failure usually occurs at the most inopportune time! Locomotive failures, minor derailments, inoperative switches and signal malfunctions will occur and result in unplanned delays. While a certain number of equipment delays must be expected, too many would indicate that maintenance needs improvement.

Major Accidents: While these represent a risk of business, their impact can be minimized by careful disaster planning and thorough investigation for each cause. By knowing the actual cause, future accidents can be prevented through changes in operating practices, maintenance or training.

Acts of God: Floods, ice storms and other weather related occurrences can wreak havoc with schedules. Where financial resources are available, some of these can be reduced by line relocation or other improvements.

Determining Costs

Direct costs to the delayed train are relatively simple to determine. If crew costs are increased by the delay, the direct increase in overtime or arbitrary payments created by the delay would be chargeable. However, many delays will not actually increase payments to the crew involved, but may increase expenses by requiring deadheading of other crews or payment for increased "away from home" time.

Locomotives are generally in limited supply, and are frequently scheduled on a system basis. Hourly cost of operation and ownership cost can generally be calculated. Not only is the cost of the locomotive chargeable to the delay, but if the delay results in the locomotive sitting idle at the next terminal, that time would also be chargeable. Locomotive utilization hours that are lost as a result of a delay are never recoverable.

Car costs can be represented by the car hire cost of the particular cars involved. Not only should the delay time be calculated, but any additional costs created by missed connections or late delivery to other carriers should also be calculated. If, for example, the delayed cars are held an additional 12 or 24 hours because of a missed connection, the additional time, switching costs and track storage costs would all be chargeable to the delay.

The intangible costs are much more difficult to evaluate. Not only do each car and locomotive have a tangible hourly cost for maintenance and capital, it also is a revenue producing tool. The earnings potential of the time lost should be considered, especially when costs are being developed to justify capital improvements to the line. Where delays result in lost time at terminals because congestion requires holding trains out, or results in additional terminal costs, this should be included.

The effects of delays on customer relations are difficult to quantify. Many of today's transportation contracts have service reliability clauses. Not only does this create additional costs if delayed cars must be given special handling, but non-priority traffic may be delayed because of this special handling. Frequent delays that affect the customer's ability to maintain reasonable inventory levels or that interfere with production or sales, can result in the traffic being diverted to other rail or highway carriers. Inventory costs are of prime concern to all our customers, and the "just in time" inventory philosophy requires a regular, dependable delivery of needed parts.

Where traffic is in jeopardy because of delays, the potential loss in revenue can be used to justify improvements. Similarly, if new traffic can be gained by better performance, these revenues can be used to justify the cost of delay prevention.

Analysis of Delays

Once delays, and locations where delays are concentrated, have been determined, a detailed analysis of the causes can be conducted. Computer simulation of train operation using a mathematical model of the train route or network will permit comparing the operation of individual trains or an entire network using different schedules. The operation can be rerun changing one or more variables and the impact of the change evaluated knowing exactly what was changed.

Simulations are valuable tools to evaluate large amounts of information needed to identify improvements. Often they represent the only practical method of testing operational changes. The simulation program used must accurately model the real conditions to produce usable information. If the model is not correct, the analysis will produce incorrect solutions. To validate the model being used, comparisons with actual field results should be used to calibrate it.

Conclusions

Analysis of the cost of delays to train operation can be used to accurately evaluate alternatives for improved train operation and service to customers.

PUBLISHED AS INFORMATION AMERICAN RAILWAY ENGINEERING ASSOCIATION COMMITTEE 16—ECONOMICS OF RAILWAY ENGINEERING AND OPERATIONS

Chairman: Johnny M. Johnson
William J. Matthews and Robert W. McKnight*

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Part 1:	Organizations
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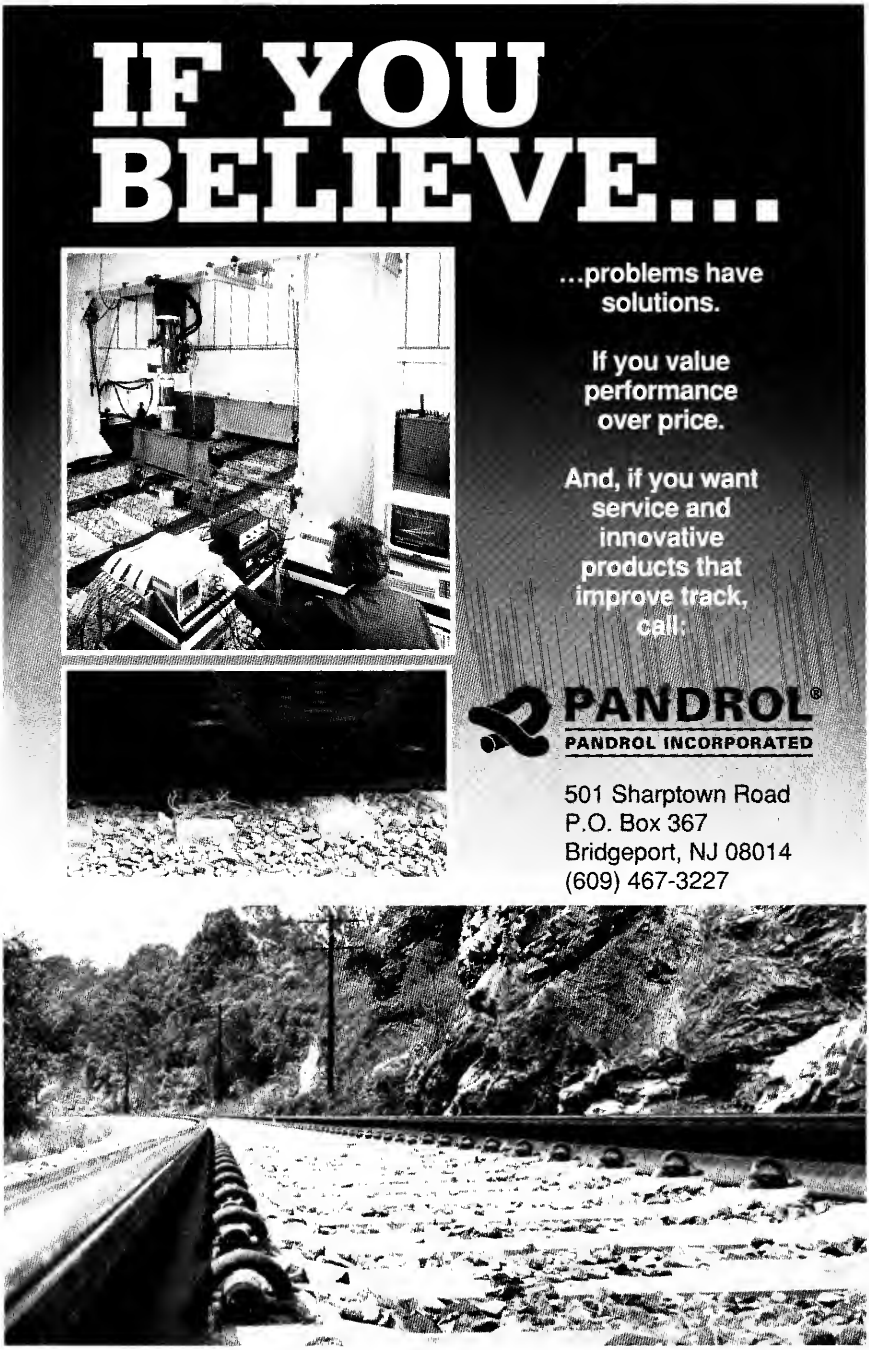

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A COMMUTER RAIL PLAN FOR METROPOLITAN NORTH GEORGIA

By: Stanley G. Feinsod*

(This is a synopsis of the presentation made to the AREA Annual Technical Conference in Chicago, IL in March 1995.)

Commuter Rail/An Investment in the Future

Clean, fast commuter trains attract passengers, give a reliable and anxiety-free ride, ease highway congestion, protect mobility, and improve the quality of life. This is happening in a growing number of metropolitan areas. Miami, Northern Virginia, San Diego, Los Angeles—all have recently started new commuter rail systems from scratch. They've joined Chicago, New York, San Francisco, Boston, Philadelphia, Montreal, Toronto, New Jersey, Maryland, and Connecticut in expanding and improving commuter railroads as an alternative to ever-more-crowded highways.

While the markets and demographics vary from place to place, one simple fact stands out: *Everywhere in North America where commuter rail now exists, it is thriving and being expanded.* Commuter rail is the transportation system of choice for growing major metropolitan areas. Now Georgians have an opportunity to take charge of future mobility by creating a network with six commuter lines serving as many as 40 stations, which will link fast growing communities in the suburbs with the employment centers in the heart of the region.

Building a commuter rail system is an investment in the future. Commuter rail costs much less than equivalent highway capacity and helps to improve air quality. A commuter rail system will benefit the regional economy, create jobs, and increase private productivity.

U.S. Metropolitan Areas Planning or Operating Commuter Rail Systems

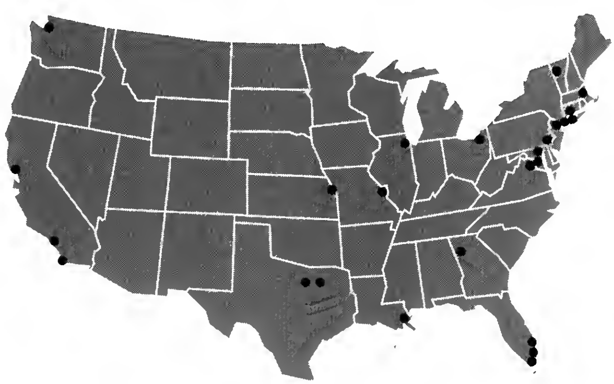

Figure 1. Joining the systems already in operation, planning is underway in Atlanta, Cleveland, Seattle, St. Louis, Dallas, and Fort Worth, and numerous other cities throughout the U.S.

*Senior Vice President, LS Transit Systems

Commuter Rail Plan/A Case of Managing Growth

Employment and population growth will place increasing strain on the existing highway network and the quality of life in the towns surrounding the center of the Northern Georgia region. Highways will always be important arteries for travel within the region, but because of the increased use, average speeds will drop below 25 mph. Adding lanes on existing roads is prohibitively expensive, and extremely difficult from the environmental and community points of view.

As population continues to grow in the suburbs, jobs continue to be added in downtown, midtown, and other inside-the-Perimeter (I-285) employment centers. This will continue to increase the number of people commuting to the center of the region, as the highway network reaches its practical limits.

A solution to this problem is the rediscovery of the commuter train running on existing freight lines into and through the heart of the region. Could this work in Metropolitan North Georgia? That was the purpose of the Georgia Department of Transportation Commuter Rail Plan—to determine whether commuter rail could be an effective way of enhancing the mobility of the entire Metropolitan North Georgia region.

The answer is *yes* if a practical and economic plan can be developed using existing rail corridors to implement new passenger services to a select number of travel corridors within the region. This report outlines the steps taken to develop such a Commuter Rail Plan for Metropolitan North Georgia.

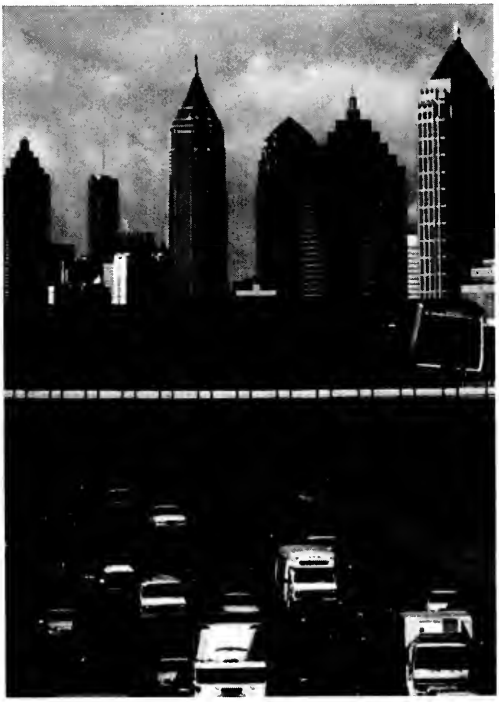

Figure 2. The metropolitan Atlanta area would be the hub of a new commuter rail network for the Northern Georgia region, joining numerous other cities in the U.S. now operating or planning new commuter rail services.

Project Blueprint/A Strategy for Metropolitan North Georgia

The Commuter Rail Plan identifies six lines as having solid ridership potential, contributing a high percentage of fare revenue to overall operating cost, and having a low pre-rider cost.

The three initial lines, expected to be available in the year 2000, are Athens, Senoia, and Bremen (a total of 158 miles), linking 20 stations in 12 counties. Approximately 6,300 riders would be carried over the three lines, totaling 12,600 daily trips. These lines are proposed to be developed first because they represent a geographic balance, a mix of existing operational freight railroads, and a manageable portion of the overall investment.

The estimated cost to build this first-phase system is $243 million, with operating assistance of $9 million annually.

The three lines proposed for a second phase in 2010 are Madison, Gainesville, and Canton (a total of 164 miles), linking 19 stations in 10 counties. These lines would carry 7,850 riders, for a total of 15,700 daily trips. The capital cost to build this second phase system is $265 million, with an additional operating assistance of $8 million annually.

Current recommendations call for diesel-powered push-pull bi-level trains, similar to those used in Chicago, Miami, San Diego, Los Angeles, and Toronto.

Figure 3. Metrolink commuter train in California.

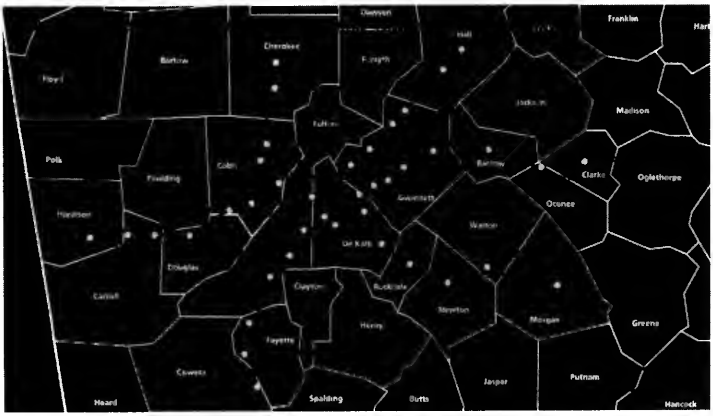

Figure 4. Metropolitan North Georgia Proposed Commuter Rail Plan.

Metropolitan North Georgia Commuter Rail Plan/The Approach

How the Plan was Developed

Defined Demand—Ridership potential was examined and documented by the best available economic and travel-demand forecasting methods.

Analyzed Railroad Operations—Each of the twelve lines were analyzed to determine the track, signal, and other right-of-way improvements required to permit reliable freight and passenger operations.

Ranked Corridors—Twelve existing railroad lines were ranked according to condition, volume of freight traffic, potential ridership, and costs of implementing service.

Refined Options—Of the twelve corridors, six emerged as strong enough to support commuter rail service.

Designed Start-up Strategy—A start-up strategy was designed to allow three lines to begin running in the year 2000 and three more in 2010.

Identified the Benefits—These six lines will cost $120 million less to build than equivalent highways and will save $23 million per year in operating, pollution, parking, vehicle, and time costs.

The study to decide whether commuter rail is right for the Atlanta metro area began in 1992 when the Georgia Department of Transportation began to look at potential corridors. To keep the metro area's vitality and economic edge, DOT and other public agencies and officials started with the idea that a multi-modal approach was needed to chart long-range transportation planning and investment.

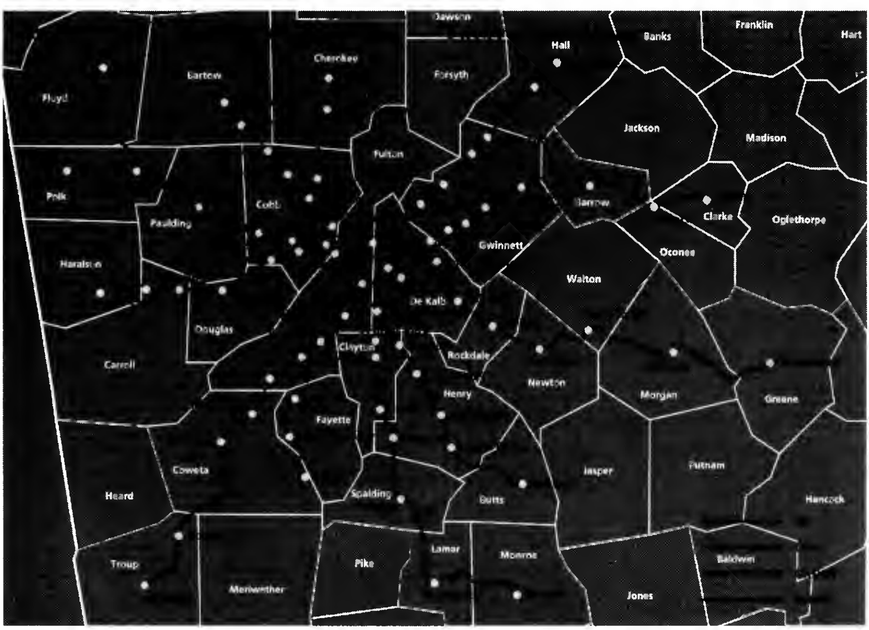

Figure 5. Twelve existing rail lines were analyzed for commuter ridership and overall feasibility—Athens, Greensboro, Rome, Jackson, Forsyth, Canton, Senoia, LaGrange, Cartersville, Bremen, Cedartown, and Gainesville.

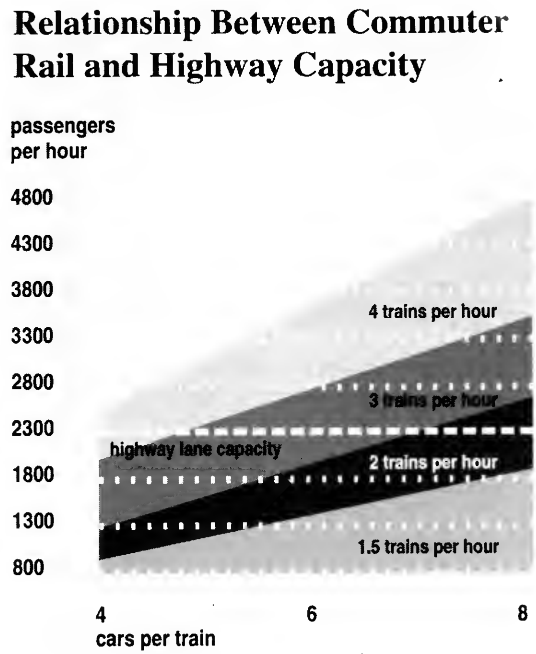

Figure 6. The proposed commuter rail train and infrastructure investment and proposed level of service can provide capacity equivalent to or greater than an interstate highway lane.

In 1993, Georgia DOT contracted with a team of consultants led by LS Transit Systems, Inc., to conduct a commuter rail feasibility study. Among the issues studied were:

- existing commuter patterns,
- population and employment trends,
- the need and demand for commuter rail,
- the cost of creating a system (designing and building stations, buying trains, modifying tracks and signals), and
- the cost of operating it.

Carving out entirely new rights-of-way for any means of transportation—railroad, highway, rapid transit, or light-rail—is prohibitively expensive where it is even possible. Investing in a commuter rail system that can use existing freight lines makes sense because it is less difficult, costly, and time consuming than creating all new corridors.

The Right Conditions for New Passenger Rail Service

Existing Corridors—An existing network of high quality freight track is in place, feeding into the center of the region.

Intermodal Integration—An existing rapid transit and bus system would allow rail commuters to make connections from the train to and from their jobs.

Figure 7. Growing congestion on existing highway corridors emphasizes the need for commuter rail service.

Local Commitment—Public policy commitments have been made to invest in the Five Points area, the proposed downtown commuter rail station and transfer complex.

Long Term Planning—The area has a history of making farsighted investments to preserve transportation mobility—evidenced by the extensive highway system, Hartsfield airport, and the MARTA rapid transit system.

The Regional Picture/Socioeconomics and Travel

Population and Employment

In contrast to many other U.S. metropolitan regions which experienced little growth during the 1980's, the population of the 50-county Atlanta metropolitan area rose by 30 percent from 1980 to 1990, while employment grew by 50 percent. Population projections for the year 2010 indicate a rise of some 50 percent over 1990 levels. Employment trends for the same year indicate a level 55 percent greater than that of 1990.

Even more important than the size of the projected growth is where that growth will take place—population is growing in the suburbs, 15 to 30 miles from the center of Atlanta.

Employment is growing in outlying zones, but also in the traditional downtown, midtown, airport, and Lenox areas, and other transit-accessible areas. *This combination of suburban growth and continued employment in the center has high potential for commuter rail.*

Existing Public Transportation

Public transportation in the metropolitan area is limited in coverage. In Dekalb and Fulton counties, the Metropolitan Atlanta Rapid Transit Authority (MARTA) operates a bus and rapid-transit network carrying about 450,000 passengers a day. Other local public agencies provide local transit service in Athens, Cobb County, and Gainesville. Cobb County also provides connecting bus services to MARTA in Atlanta's midtown area.

Travel Patterns

Three travel behavior surveys formed the basis for learning about residents' existing commuter patterns and preferences. A 1990 Atlanta Regional Commission survey of 2,500 households in the seven-county ARC region provided the most current source of public-transportation travel data from that area. This was supplemented by a 1993 mailback-telephone survey in which responses were gathered from 2,000 households in 43 surrounding counties.

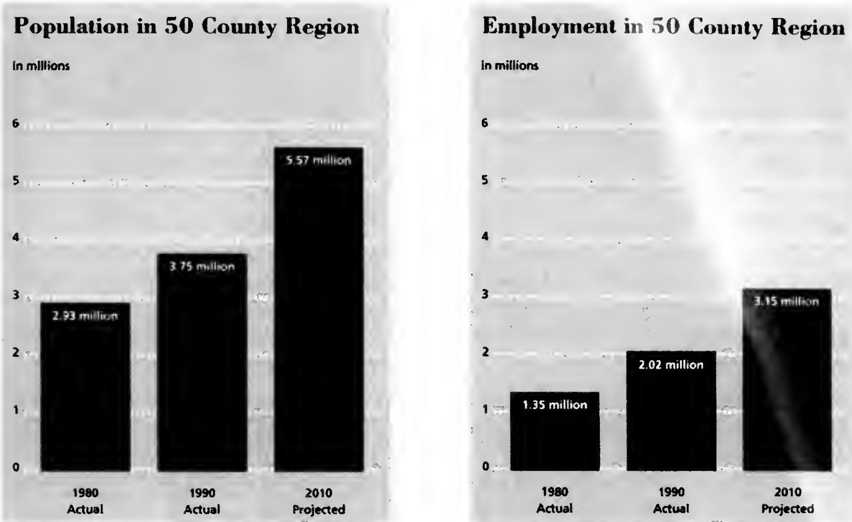

Figure 8.

Figure 9. The "expressed preference" survey gave respondents theoretical scenarios regarding commuter rail travel time, costs, parking, and other factors, to determine if respondents would choose commuter rail in comparison to their present mode.

An "expressed preference" survey was also conducted. People were shown an interactive video presentation about commuter rail and asked to compare it to their current trip downtown and decide if they would choose the train, or stay with their current way of traveling.

Market Research/How Many Will Ride

The Four-Step Modeling Process

Trip Generation—Identified where each journey begins.

Mode Split—Determined existing preference for various means of travel, taking into account time, cost, and convenience.

Trip Distribution—Identified where each journey ends

Trip Assignment—Determined how many trips begin and end at each station.

Sophisticated computer based travel demand models were applied to predict how many riders would use commuter rail service. The models incorporate information about travel habits on the specific corridor being studied, such as length of trip, existing preferences, and commuting patterns. Although the models are based on information and interview responses from metropolitan-area residents, the results were cross-checked against those of other commuter rail corridors to ensure their reasonableness.

The area encompassed 50 counties that were divided into nine regions and 1,291 zones. The models were run using several scenarios to evaluate each of the 12 lines independently. Only work trips (to and from jobs) were counted, excluding off-peak, shopping, and recreational riders. For example, by the year 2010, the model found the range of riders attracted went from a high of 8,900 passengers per day on the Athens line to a low of 2,700 passengers per day on the Jackson and LaGrange lines.

Figure 10. Ridership estimates for daily work commuting indicate significant demand for many of the corridors evaluated.

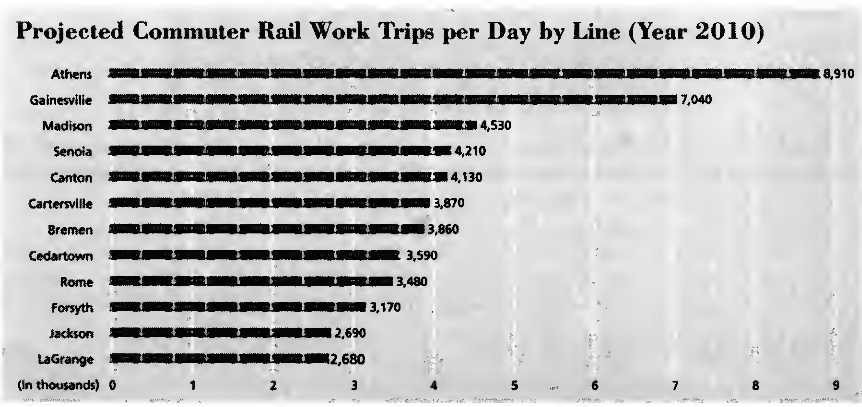

Figure 11.

Comparing the Lines

The 12 lines were analyzed for ridership potential and for the costs of adding passenger trains to the freight traffic on each route.

Much of the required investment in right-of-way improvements is to assure that freight trains and passenger trains can be accommodated without delays and with enough capacity for future growth. A detailed operations analysis of each line determined what right-of-way investments would be needed to assure reliable passenger and freight service. The goal of the technical work was to:

- define each line's strengths and weaknesses,
- determine if lines competed for riders from common areas,
- learn whether significant ridership could be attracted,
- identify which station locations were most attractive, and
- identify the costs of improving the right-of-way.

For each line, capital costs were estimated to compare improvements to railroad infrastructure (track, signals, structures, stations, and parking) and rolling stock purchases. Joint facilities that would serve all lines were not included. Capital costs ranged from $48 million for the Bremen line to $123 million for the Rome line.

Corridor Capital Costs
(000's of 1994$)

Corridor	RR	Infrastructure	Rolling Stock	Total
Athens	CSX	$53,003	$55,000	$108,003
Madison	CSX	$36,798	$35,600	$72,398
Jackson	NS	$25,054	$27,200	$52,254
Forsyth	NS	$47,024	$28,600	$75,624
LaGrange	CSX	$33,425	$27,200	$60,625
Senoia	CSX	$16,505	$35,600	$52,105
Bremen	NS	$15,575	$32,800	$48,375
Cedartown	CSX	$59,818	$31,400	$91,218
Rome	NS	$91,164	$31,400	$122,564
Cartersville	CSX	$78,402	$32,800	$111,202
Canton	GNRR	$55,908	$34,200	$90,108
Gainesville	NS	$28,745	$44,000	$72,745

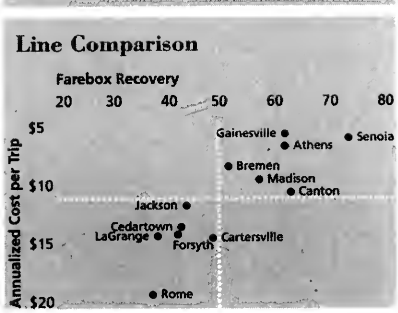

Figure 12.

Other factors which contributed to an overall cost/benefit comparison included *farebox recovery and annualized cost per rider, which takes into account both operating and capital costs.* The farebox recovery index ranged from 73 percent for the Senoia line to 37 percent for the Rome line. Other commuter systems in the United States typically cover 40 to 60 percent of costs. The Gainesville line showed the lowest annualized cost per rider per trip at $5.92, while the Rome line showed the highest at $19.12.

When the 12 corridors were compared with all of these factors considered, six lines stood out as being much stronger than the rest: *Senoia, Athens, Gainesville, Madison, Canton, and Bremen.*

The Plan for Operations

Simulating the Proposed Service

A set of basic operating standards were established to compare the lines.

Service frequency was assumed to be three inbound morning trips (arriving between 7:15 a.m. and 8:30 a.m.), one midday round-trip, three outbound late-afternoon trips (leaving between 4:30 p.m. and 6:00 p.m.), and an evening round-trip.

In peak periods, the trains would run 40 minutes apart and stop at all stations. Trip times assumed speeds of 40 to 60 mph most of the way. Run times ranged from a low of 54 minutes from Semoia to Atlanta (Five Points) to 1 hour 47 minutes from Forsyth to Atlanta. One-way fares were assumed to range from $1.50 to $6.00, depending on distance. It was assumed that rail commuters could transfer free to MARTA transit or bus service.

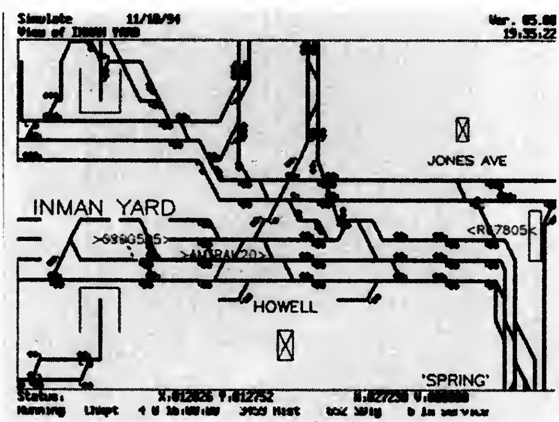

Figure 13. A computer model was developed to simulate the actual operation of the railroad as trains move over the track under the control of the signal system. The impact of the introduction of passenger trains was studied and changes to the signal and track systems were introduced to assure that capacity and reliability were maintained.

Distance Based Fare Structure

Distance	0–10	10–15	15–20	20–25	25–30	30–40	40–50	50–60	60–70
Fare	$1.50	$2.00	$2.50	$3.00	$3.50	$4.00	$4.50	$5.00	$5.50

Figure 14.

Local station location assumptions were based on available sites that were accessible from streets and highways. Stations would have bicycle storage, drop-off points for arriving passengers, bus stops, and adequate parking.

A downtown transfer station was assumed at Atlanta's Five Points to accommodate riders who will use MARTA trains and buses to complete their trip. This is the site of the proposed Multimodal Passenger Terminal.

Permanent maintenance and storage facilities would be needed in Atlanta and at each outlying terminal. Three trainsets would be required for each line, each set consisting of a diesel locomotive and three to six bi-level commuter coaches, with a cab control car allowing the train to be controlled from either end to eliminate the need for turning the equipment after each trip.

A Strategy to Launch Commuter Rail

While six lines are good candidates for commuter service, the cost and complexity of launching such a large network all at once make a good case for a staged approach over a 10- to 15-year period. By beginning with three lines and adding three others later, resources can be conserved and a new travel system can be introduced one step at a time.

Phase 1 envisions starting the Athens, Senoia, and Bremen lines first—by year 2000—providing a geographic mix, diversity of host freight railroads, and a relatively low initial capital investment. These three lines would serve a downtown transfer station and 20 stations in 12 counties, carrying more than 6,300 riders daily (12,600 daily one-way trips). Capital cost would be $243 million, and annual operating assistance would be $9 million.

In Phase 2, the lines to Madison, Gainesville, and Canton would be added, with the first trains running in year 2010. These lines would carry more than 7,850 passengers a day (15,700 daily one-way trips). Capital cost would be $265 million, and the additional annual operating assistance needed would be $8 million.

In its final form, the system would service 40 stations in 18 counties. The estimated population of those 18 counties by year 2010 is 4.3 million people, more than half of the estimated total population of Georgia in that year.

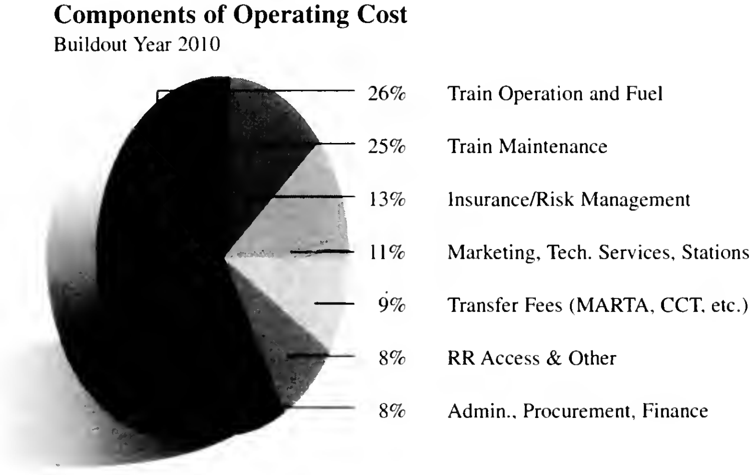

Figure 15.

Ridership and Cost Statistics

	Phase 1 Year 2000	Growth to Year 2010	Phase 2 Year 2010	Six Lines in Service
Annual Trips (000's)	3,165	[illegible]	3,925	[illegible]
Operating Costs (000's of 1994$)				
Annual Revenue	$10,130	[illegible]	$12,400	[illegible]
Annual Costs	$19,320	[illegible]	$20,340	[illegible]
Annual Support	$9,190	[illegible]	$7,940	[illegible]
Capital Costs (000's of 1994$)	$243,200	[illegible]	$265,250	[illegible]

Figure 16.

Focus on Implementation/Steps to Action

What Commuter Rail Can Bring to Northern Georgia

Preserving Mobility—It preserves mobility while fostering continued economic growth.

Regional Benefits—It can unite the regional economic development strategy and protect the vitality of the region's core by maintaining its accessibility.

Private Sector Impacts—Investments in commuter rail improve private sector productivity at twice the rate of spending on additional highway lanes, underscoring the positive impact public policy can have on private enterprise. For the individual commuter/employee, time not spent in traffic congestion is productive time.

Cost Savings—Commuter rail costs only one-third as much as building equivalent highway capacity.

Before the first ticket can be sold, public policy leaders will take several critical steps which fall into three key areas:

- Define responsibility for planning, designing, constructing, operating, marketing, and maintaining the system.
- Establish a long-term financing plan, acceptable by state, regional, and local officials, including potential statewide transportation development sales tax.
- Finalize agreements with the freight railroads that define operational jurisdiction as well as the right to make and maintain investments on rights-of-way, and operate commuter trains.

Figure 17.

Net Annual Benefit of Commuter Rail Service

Benefits	Annual Value
Value of Construction Costs Avoided in 2010	$12.4 million
Cost Savings of Purchasing Rail Vehicles vs. Automobiles	$7.8 million
Operating, Maintenance, Parking and Value of Increased Usefulness of Time	–$3.7 million
Value of Emissions Reduction	$7.0 million
Total Net Annual Benefit	**$23.5 million**

Figure 18.

The Project Team

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THE NEUTRAL TEMPERATURE FALLING OF CONTINUOUS WELDED RAIL: DUE TO PLASTIC ELONGATION IN ENTIRE CROSS-SECTION

By: Liu Fucang*
Cooperators: He Qubo, Chen Juncai, Yang Guangcong, Guo Mei

Abstract

A destressing of a continuous welded rail (CWR) offers an opportunity to obtain the length of the CWR in force free state. By contrasting these lengths of a CWR at different times, it will be observed that a CWR lengthens itself plastically in service. This plastic elongation is in the entire cross-section (not only in the top layers of the rail head), and causes the decreasing of the average neutral temperature of a CWR on its whole length, which is called "neutral temperature falling". In over one hundred CWR's on Liuzhou Railway, China, the largest neutral temperature falling from new rail being laid until the 1st destressing exceeded 10°C. These CWR's were laid in 1979–82 and destressed in 1988–90. Several months later, after the 1st destressing, a part of these CWR's were destressed once more. In the period between the two destressings, a new "neutral temperature falling" was born and grew rapidly to the former level. Hence, the author of this paper advanced an explanation of these phenomena; ie: that a non-typical creeping of steel takes place in CWR in service.

Introduction

With regard to the shift of the neutral temperature—another name is force (or stress) free temperature—of continuous welded rail (CWR), the general research way is based on measuring the longitudinal strain at some points, ie: cross-sections, of a CWR. Many such measurement results have shown that the neutral temperature tends to shift downward. However, the longitudinal movement of a section of a CWR can lead to a reduced neutral temperature in a section and an increase in its neighbor. But, by observing shift of the average neutral temperature of a CWR on its entire length, the law of the neutral temperature shift may be seen easier and more accurately.

Along these lines, the engineers of the Chinese railway have in the past done many works. This paper mainly introduces the current research efforts by Liuzhou Railway Administration, China.

Neutral temperature falling

Average neutral temperature

For required track maintenance in China, the railway administrations destress a great number of CWR's each year. During the process of destressing, some steps are used to help a CWR freely contract or expand, such as to place steel pipes between the rail and concrete-ties, to vibrate the rail, etc. After that, the remaining longitudinal forces in the CWR at this moment are small. The forces are generally treated as zero at the free end of the CWR, and tend to increase at a definite average progressive rate along the CWR to the fixed end. Hence the unreleased length attributed to the remaining longitudinal forces, ie: the unrealized displacement of the CWR free end, can be calculated. Utilizing the length at this moment and the remaining forces of a CWR, the length of the CWR in force free can be obtained. The measured temperature of the CWR at this moment is the force-free temperature corresponding with the length of the CWR in force free. Now this length and this temperature constitute a basis. On this basis and by utilizing the formula $\Delta L = \Delta T^{*}\propto^{*}$, another force-free temperature corresponding with another length of this CWR can be easily known. Therefore, the T_n

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of the CWR constrained just before destressing or relayed on the final step of the destressing process can be calculated. In this paper, the T_n is the average neutral temperature of a CWR on its whole length. Thus, to engineers, a destressing provides an opportunity to obtain the length of a CWR in a force-free state. As a result of that, since the destressed CWR's on Chinese Rail are numerous and many of them have even been destressed more than once, some laws of CWR have presented themselves.

Neutral temperature falling

The laying temperature T_L of a CWR when new can be regarded as initial T_n. In comparing T_L of a CWR to the T_n just before destressing of this same CWR, the shift of T_n from new rail being laid can be seen. On this matter, the China Academy of Railway Sciences and the Xiuxian Maintenance of Way Section (in Anhui province), respectively, did much work around 1980. Their work showed that the average neutral temperature tends strongly to shift downward. In other words, the decreasing of the average neutral temperature is a general phenomenon, although the length of a constrained CWR has not changed from the new rail. Hence, the Xiuxian Section named this kind of decreasing of the average neutral temperature as the "neutral temperature falling", to distinguish it from the one due to rail-creep, ie: an elastic displacement of CWR induced by mechanical or thermal loads. In 1986, the China Academy of Railway Sciences proposed to regard the neutral temperature falling from T_L as −8°C on an average. At that time, the remaining longitudinal forces of a CWR in destressing were regarded as zero. With respect to the cause of the neutral temperature falling, they accepted the well known explanation of Derby, British Rail (BR).

Samples on Liuzhou Railway

As aforementioned, the Liuzhou Railway Administration destressed over 150 CWR's, ie: 150 strings of CWR, in south of Guangxi province in 1988–1990 because of the necessity of sifting ballast. These CWR's were laid in 1979–1982 when they were new, and the length of each CWR of them is about 700–1500m. This was their first destressing. The railway involved with these CWR's is a general line with a nearly equal volume of traffic in both direction. The topography along the road is smooth. Obviously, the destressing of these CWR's could be utilized to test the neutral temperature falling. Accordingly, some engineers of the administration calculated the shift of T_n from new rail being laid to the first destressing for each one of 123 CWR's. To determine the unreleased length of a CWR in destressing, they performed many measurements, utilizing the same CWR's. The calculation formulas are listed below:

$$T_r = T_m - \frac{\Delta L_1 + \Delta L_2}{\propto L} \qquad \text{(Eq.1)}$$

$$\Delta T = T_r - \frac{\Delta L_c}{\propto L} - T_L \qquad \text{(Eq.2)}$$

Here: T_r = average neutral temperature just before destressing. T_m = measured temperature of CWR at moment concurrent with ΔL_1.ΔL_1 = automatic displacement of CWR free end in destressing. To expand + and to contract −, similarly hereinafter:

ΔL_2 = unrealized displacement of CWR free end in distressing owing to remaining longitudinal forces.

ΔL_c = difference between the length of CWR just before destressing and the laying length.

$\propto$ = coefficient of thermal expansion.

L = CWR length, well approximate by the laying length L_o.

ΔT= shift of average neutral temperature.

T_L = laying temperature of CWR being new.

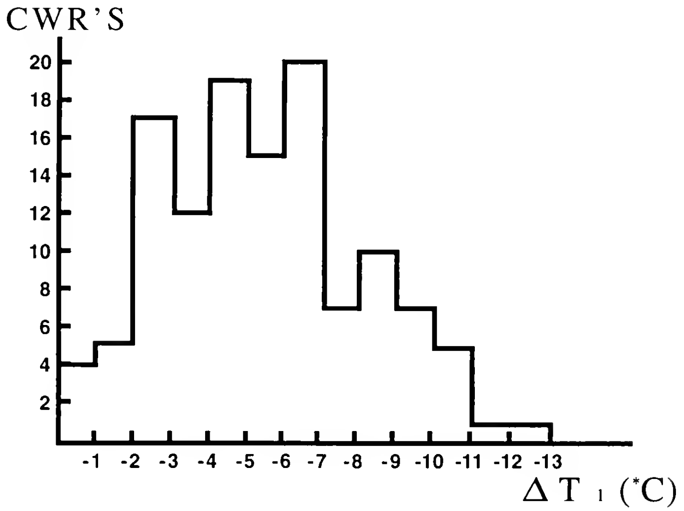

Figure 1. Distribution of ΔT_l among 123 CWR's.

Based on these calculations, all ΔT's are below zero. The average value of them is –5.4° C. Their distribution is shown in Figure 1, in which the ΔT is displaced by ΔT_l. This will be explained later.

Of course, the value of a ΔT must be under the influence of the errors of parameters in Equations 1 and 2. But the synthetic error of these parameters, except T_L, was only about 1.1° C. As to T_L, its errors can not have been one-way and as large as ΔT. Therefore, if all errors had been zero, the ΔT's would have still been as Figure 1 on whole.

Figure 1 certifies once more surely the existence of neutral temperature falling from new rail being laid. Although the study method is approximate, its precision level is sufficient to demonstrate results.

Cause of neutral temperature falling

The foregoing works are based on the force free state of a CWR in destressing, ie: the instantaneous T and L of the CWR in a force free. By studying this principle further, the cause of the neutral temperature falling can be discovered.

Plastic elongation of CWR

Assuming that the length of a new laid CWR is L_o. According to the premise of Equations 1 and 2, the length of the CWR in force free in destressing is $L_o + \Delta L_c + \Delta L_1 + \Delta L_2$, when the measured temperature is T_m. With F assumed to be the thermal longitudinal force, therefore:

$$L(T = T_m,\ F = 0) = L_o + \Delta L_c + \Delta L_1 + \Delta L_2 \qquad \text{(Eq. 3)}$$

Provided the CWR at this time can expand or contract with no resistance, and using the basic formula $\Delta L = \Delta T * \propto * L$, the following equations are clear:

$$\begin{aligned} L(T = T_r,\ F = 0) &= L_o + \Delta L_c \\ L(T = T_L + \Delta T,\ F = 0) &= L_o \end{aligned} \qquad \text{(Eq. 4)}$$

As the $\Delta T < 0$,

$$L(T + T_L,\ F = 0) = L_o + |\Delta T| * \propto * L_o \quad \text{(Eq.5)}$$

Equation 5 expresses the situation of a CWR just before destressing. But when a new CWR is being laid, the longitudinal force on average can be regarded as zero. Therefore, at that time

$$L(T = T_L,\ F =) = L_o \quad \text{(Eq.6)}$$

By contrasting Equation 5 with Equation 6, it can be seen that the property length of a CWR has grown and increased $|\Delta T| * \propto * L_o$ in service. Here the property length of a CWR represents the length of the CWR in force free and in a certain temperature. Hence, it follows that the $|\Delta T| * \propto * L_o$ is a plastic elongation.

Since there is no distortion in the cross-section of a service-worn CWR, except the metal in the top layers of the rail head has slightly flowed relatively, it is apparent that this plastic elongation took place in the entire cross-section.

The Equations 4 and 5 show that the plastic elongation causes the neutral temperature falling.

Mechanism of neutral temperature falling

From the preceding segment, the property length of a CWR is:

$$L(T = T,\ F = 0) = L_o + (T - T_L) * \propto * L_o + |\Delta T| * \propto * L_o \quad \text{(Eq. 7)}$$

Among the three parameters T, L and F, only two of them are independent. Thus:

$$L(T = T,\ F \neq 0) \neq L_o + (T - T_L) * \propto * L_o + |\Delta T| * \propto * L_o$$

In this case, the elastic strain on average is:

$$\in e = (L - L_o) / L_o - (T - T_L) * \propto - |\Delta T| * \propto \quad \text{(Eq. 8)}$$

the $|\Delta T| * \propto$ is a plastics tensile strain

$$\in_p = |\Delta T| * \propto > 0$$

It can be seen in Equation 8 that the plastic tensile strain causes a corresponding elastic compression strain when the CWR is constrained. For example, if:

$$(L - L_o) / L_o - (T - T_L) * \propto = 0$$

Then the $\in e$ in Equation 8 is:

$$\in_e = - \in_p$$

This is why the neutral temperature falling and the plastic elongation $|\Delta T| * \propto * L_o$ are invisible while the CWR is constrained. Only when the CWR is destressed near $\in_e = 0$ ($F = 0$), the plastic elongation appears through $L - L_o > (T_m - T_L) * \propto * L_o$ (see Equation 7)

In fact, in the samples on Liuzhou Railway given above, there were a number of CWR's whose $T_m < T_L$, but $L - L_o > o$, in destressing, ie: $\Delta L_1 > 0$, $\Delta L_c \approx 0$. This is a powerful certification of the plastic elongation. Similarly in the past, people noticed (but did not understand) the plastic elongation first, then understood the ΔT.

Equations 8 and 9 demonstrated that the plastic elongation causes the neutral temperature falling.

Second "neutral temperature falling"

On the Liuzhou Railway, to avoid CWR buckling in the period of ballast sifting, the relaying temperature (on mechanics) in 1st destressing must arrive at about 40° C–50° C by machine lengthening. Later in 3–6 months, to get the neutral temperature to come back into normal range (25° C–35°C), the second destressing has to be done for a part of CWR's previously destressed once.

In the 2nd destressing done in autumn 1988, a new plastic elongation was discovered growing immediately after the 1st destressing. The discovery came from contrasting L and T in force free in the 2nd destressing with the 1st destressing. For instance, when the neutral temperature of a CWR came back from 45° to 25°, however, the CWR contracted itself as much as corresponding with only 12°, as if it had been pull-lengthened by machine, although the tensile stress in the rail was only 30 MPa. These phenomena have been repeating since early 1988. The neutral temperature falling corresponding with the new plastic elongation, named ΔT_2, was on a level with ΔT_1 occurring from new rail being laid until the 1st destressing as shown in Figure 1.

But at that time, the engineers of Liuzhou Railway did not understand that the elongation is plastic, because the aforecited cause and mechanism of neutral temperature falling had not been found yet. The theory accepted then was the explanation on "rolling out" and " residual stresss conversion" proposed by British Rail. This theory can explain ΔT_1, but not ΔT_2. Accordingly, those engineers doubted the truthfulness of the ΔT_2. Then some engineers (the author of this paper was one of them and in charge), started the current research.

The formulas to determine ΔT_2 are also those Equations 1 and 2. But here: T_r is just before the 2nd destressing; ΔL_c is between the two destressings: T_L is displaced by T_1-relaying temperature in the 1st destressing; other parameters are in the 2nd destressing.

An important point of the research is the remaining longitudinal forces in destressing. In this case, a disassembled CWR is on steel pipes (one every 10m) placed on concrete-ties on which the rubber pads have been removed. In addition, there are some production details which will decrease or increase the forces. In China in the 1970's, people regarded the forces all as zero in conditions at that time. Later, in CWR regulations in 1988, the China Academy of Railway Sciences determined the progressive rate of the forces as 4–6kg/m. However, for confirming ΔT_2 to be true or false, the Liuzhou researchers created a few methods to measure—just in destressing—the ΔL_2. The measurements showed that 4–6kg/m is appropriate for the average progressive rate of the remaining longitudinal forces on certain conditions. Then, the researchers calculated ΔT_2. For reliability of ΔT_2, in calculation, most ΔL_2's were from the measurements themselves and the other ΔL_2's were from the progressive rate of the remaining forces based on measuring in the same conditions as theirs. Later, in passing, the researchers calculated the ΔT_1 in Fig. 1.

From the calculation results, all ΔT_2 were below zero. The distribution of ΔT_2 among some CWRs's is shown in Figure 2. The synthetic error of ΔT_2 in Figure 2 was about 1.2°C. Hence, these tests proved that the existence of the neutral temperature falling beginning from relaying is true. Just judging from this, researchers gave up doubt about ΔT_2 and affirmed that the cause of the neutral temperature falling is the plastic elongation in the entire cross-section of a CWR.

Some examples of the ΔT_1 and ΔT_2 of that same CWR are shown in Table 1. Obviously, the length of time between the two destressings had a certain influence on the dimension of ΔT_2.

Steel creeping of CWR

According to experiments of the Railway Technical Center Derby BR, the "rolling out" and the "residual stresses conversion" grows rapidly in the first three months after laying new rail, continues

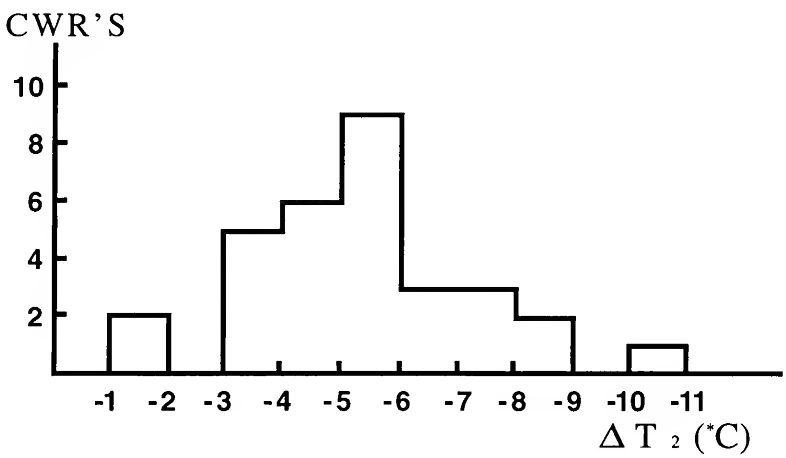

Figure 2. Distribution of ΔT_2 among 31 CWR's

Table 1. ΔT_C of some CWR

No. of CWR	T_L	ΔT_1	T_1	ΔT_2
58#L	35	–9.0	35.9	–5.9
58#R	35	–9.6	35.2	–5.5
59#L	33	–10.8	32.9	–3.8
59#R	33	–7.5	35.9	–1.1
106#L	31	–4.1	47.6	–4.6
109#R	31	–5.3	45.6	–4.8
109#L	27.5	–4.1	44.5	–5.4
109#R	27.5	–6.8	44.7	–6.8
171#L	30.1	–5.1	40.5	–4.0
171#R	30.1	–4.4	40.4	–3.0

for about a year and then "may have stabilized". The resulting "reduction in force free temperature" can be on the order of –9°C. In fact, this is just the ΔT_1 defined in this paper. Judging from the Liuzhou sample, the stabilization period of ΔT_1 must have lasted until the 1st destressing. The time was so long that the CWR's had already served nearly ten years and accumulated at least 200 MGT. However, after the CWR relaying in the 1st destressing, the ΔT_2 grows as rapidly as after laying new CWR. The substance of ΔT is the plastic elongation, so the relaying in the 1st destressing is a base line of the plastic elongation growing. On the two sides of the base line, the difference of conditions for a CWR to be in is only the rise of neutral temperature, ie: the rise of the thermal tensile stress. On the basis of the relation of the plastic elongation with time and the tensile stress, the process of the plastic elongation is just like the process of metal creeping. Therefore, the author of this paper put forward that the cause and substance of the CWR plastic elongation is the creeping of steel of CWR.

As is well known, the typical creeping of steel needs temperature at hundreds of centigrade degrees or tensile stress at hundreds of MPA. Here, the CWR is in a state with common temperature and low stress. However, here is a new factor to activate creeping, which is the rolling calendering of wheels for CWR in service. In other words, under the condition that a length element of a CWR is continually acted on its two ends by longitudinal thermal tensile forces, the vertical dynamic forces from wheels and sleepers help the metal of the element creep in the longitudinal direction. In addition, the plastic elongation strain of CWR is much smaller than the typical creeping strain of steel, hence, the explanation is reasonable that the plastic elongation is from a non-typical creeping of steel of CWR.

This explanation is concerned with the following points: 1) Many years ago, man noticed that a lead pipe hung from above lengthens itself in a common temperature for itself's weight. The research in the current century shows that hard metals also have the nature of creeping, only their creeping size is much smaller than soft metals in the same conditions. 2) Based on the Poisson ratio, it is common knowledge that the compressive forces cause the material to extend in perpendicular directions. If stresses act in a direction, and the compressive forces are dynamic and very powerful, then it is not strange that a micro plastic extension can be born in this direction. 3) The plastic elongation strain of the neutral temperature falling is mostly on a level of 10^{-5} to 10^{-4}. The level of plastic deformation for general engineering is hardly noted for it is too small. For CWR 1 km long, however, the plastic elongation is up to 100 mm. Thus it can be visible even to the naked eye, and the problem is apparent.

According to this explanation, the steel of a laid newly CWR produces longitudinal creeping under conditions of the thermal tensil stress and the calendering of wheels. The creeping elongation causes the neutral temperature falling ΔT_1. Following, the creeping speed gradually slows to stop. The period of the ΔT_1 rising is about a year. After that the ΔT_1 stabilizes so long as the aforesaid conditions roughly stabilize. When the relaying temperature in destressing is strongly higher than before, the thermal tensile stress is raised. This conversion arouses the creeeping speed to jump from zero to the same level as a new CWR when just laid. Then a new round of still creeping starts, which corresponds with the ΔT_2.

By inference from the explanation, the new creeping will stabilize as ΔT_1.

Conclusion

A CWR buckling accident is generally due to the synthesis of various factors. The reduction of the neutral temperature on the buckling section of the CWR is one of them. The neutral temperature falling influences the reduction of the neutral temperature on every section of CWR. Although the reduction of the neutral temperature was taken into account in the design against track buckle in some countrys—BR and Chinese Railway because of other factors, the size of the reduction in consideration is still discussed. The reason for this is that the two parts making up the reduction of the neutral temperature, the neutral temperature falling and the uneveness of the neutral temperature along a CWR, both can be over 10° C. When other means used in preventing buckle are weakened, the reduction of the neutral temperature will be dangerous. Besides, the neutral temperature falling may lead to some mistakes or even an accident in destressing if the engineer is not aware of it. Considering the importance of the neutral temperature falling, more research is needed.

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Front Cover: Conrail Intermodal Train TV-1, Westbound to Chicago, coming out of the enlarged Allegheny Tunnel.

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CONRAIL'S PENNSYLVANIA CLEARANCE PROJECT

By: J. C. May*

In the mid-1980s Conrail initiated its first clearance project, to develop a route cleared to 19'4" to allow 19' tall double-stacks to travel between Chicago and Kearny, NJ via the former NYC water level route west of Buffalo connecting with the former Erie Railroad route along the southern Tier of New York State counties. In 1985 Conrail cleared the remainder of the water level route from Buffalo to Syracuse and Albany, progressing south along the Hudson River into the New Jersey side of the New York City metropolitan area. As these clearances were only for 19'4", it became necessary in 1988 to initiate further clearance improvements along the route to achieve 20'6" clearance for 20'2" double-stacks. Due in large part to the restrictions posed by the tunnels in the Allegheny mountains, as well as less favorable horizontal alignment and grades, the former PRR route across Pennsylvania as well as access to most Pennsylvania cities was not a part of a Conrail high-clearance system. Although Conrail studied the Pennsylvania clearance route, economics did not support the investment at that time.

In November of 1992, the Pennsylvania State Legislature, recognizing the benefits of double-stack access to commerce for Pennsylvania and the Port of Philadelphia in particular, authorized funding a project in partnership with Conrail and the D&H Railroad to provide access for double-stack container movement to the Port of Philadelphia, which Conrail strongly supported due to the dramatic increase in domestic container traffic. A north-south route accessing the Port of Philadelphia was provided for the D&H operations. This route comes into Pennsylvania from Binghamton NY, progressing on D&H lines to DuPont Junction, from which point the D&H has overhead trackage rights on Conrail to Philadelphia via Allentown and Reading. Similarly, an east-west route was established for Conrail beginning in Cleveland, Ohio, progressing southeast to Pittsburgh and then east through Johnstown, Altoona, Harrisburg, Reading and on to the Port of Philadelphia.

See Figure 1.

By the time the formal reimbursement agreements were signed in May of 1993, Conrail had expanded the scope of the project with a spur between Crestline, OH and Alliance, OH, to bypass Cleveland for eastbound traffic from the southwest via St. Louis, as well as links between the Hagerstown, MD gateway with the Norfolk Southern and Harrisburg, and between Morrisville, PA and Conrail's major New York City terminals in northern New Jersey. Clearing these last links in partnership with the Norfolk Southern clearance work further south cleared a doublestack route from Atlanta to New York City, and eventually to Boston, which we anticipate will effectively compete with the I-95 truck corridor along the east coast.

See Figure 2.

West of Reading, where the route is utilized exclusively by Conrail, Conrail funded 70% of the project cost, with the Commonwealth funding the remaining 30%. Similarly, north of Allentown where the double-stack traffic would be D&H, the D&H funded 70% of the cost with the state funded the remaining 30%. Generally, east of Reading where both railroads utilize the route to access Philadelphia, 100% funding was provided by the Commonwealth. Total expenditures for this project on Conrail was $115 million, of which Conrail funded $77.4 million, the Commonwealth funded $32.5 million and the D&H funded $5.1 million.

*Ass't. Chief Engineer, Project Control & Special Projects, Conrail

Figure 1

Figure 2

Figure 3

See Figure 3.

The overall project schedule called for all work to be completed by the end of 1995. To achieve this goal, clearance related work had be progressed at 140 different sites, including work that we categorized into three general critical paths:

1. Overhead Bridges (39 Locations)
2. Tunnels (12 Locations)
3. Track Lowering or Undercutting Operations (54 Locations)

The first critical path, was overhead bridges. These represented a critical path in that most bridges are owned by other parties, such as the state, county, township or borough involved. As such, raising a bridge that is owned by others represents a very serious potential for delay while design is developed and approval obtained from the highway owner. More over, permission to raise the bridge itself is not always a given. Due to the Commonwealth of Pennsylvania's involvement in funding this project, considerable assistance was obtained from PennDOT at eight locations where PennDOT was planning bridge replacement in future years. In those instances, PennDOT accelerated bridge replacement which was scheduled to occur a number of years in the future by reprioritizing their twelve year capital program to replace those bridges during the clearance project time frame, thus saving the project expenditure for undercutting a bridge which would be replaced in the near future anyway.

The second path, undercutting to lower track, was critical because of the sheer number of locations in which this was the most economical solution. As we are well aware, it is not the undercutting operation itself, even if rock is involved, which is the problem in scheduling this work, but rather track time. In order to minimize conflict, productivity was improved by taking three general steps:

First, wherever possible in multiple track territory, undercutting was scheduled to hold the track out-of-service until proper depth had been reached in order to avoid the loss of productivity with putting the track back in service every night on a temporary basis.

Secondly, we used two Plasser RM-76 undercutters in tandem. By doing so, we were effectively making two passes simultaneously in a given track outage. With careful scheduling, we were able to utilize both undercutters effectively with one surfacing unit supporting the track lowering. Use of work trains to handle spoils was avoided where alternatives such as casting and preexcavating receiving areas were possible and shoulders were excavated in advance of the undercutter.

See Figure 4.

Figure 4

The *third* aspect which was very beneficial to our cause with the Transportation Department was the utilization of a track stabilizer upon completion of surfacing to accelerate restoration of track speed following the disturbance of the track structure. Conrail requires that trains operate at 10 mph until 50,000 tons of traffic have passed over the affected track and 24 hours have elapsed. Then 48 additional hours and 50,000 more tons of traffic are required before authorized speed is permitted. The stabilizer provided sufficient compaction to eliminate the need for the 10 mph requirement.

The third critical path of construction which controlled the project schedule was the tunnel clearance improvement work involving nine rock tunnels which typically had only 18'6"± clearance, which must be increased to 21 feet. While there are generally four alternate solutions: rerouting to avoid the tunnel; day lighting, raising the roof of the tunnel, and lowering the track in the tunnel, the selection of the clearance method to be implemented was in most cases determined by railroad operational requirements for track usage. An additional consideration in the tunnel solution was the solution of icing problems which previously occurred in most of the tunnels involved as a double-stack or fully enclosed auto tri-level can be just as damaged, running into a build-up of glare ice, as running into rock.

At Antis, PA, retirement of a former yard over the Main Line allowed us to replace a 400 feet long century old cut and cover tunnel with a single track three-span bridge to maintain access for customers on the north side of the Main Line.

See Figure 5.

In the eastern four tunnels on the project, White Haven, Rock Port, Flat Rock and Black Rock, the tunnels were located on single-track portions of the railroad where maintaining railroad operations was a requirement. As such, arrangements were developed with Transportation Operations to reschedule trains to operate sixteen hours a day, thus leaving an eight-hour per day window for contractor operations.

See Figure 6.

Figure 5

Contractor access for the live track tunnel work has been accomplished in either of two ways:

On two of the tunnels, the contractor established a strictly rail borne operation utilizing high rail and conventional rail car mounted equipment with car movers moving the equipment to the site from a siding and positioning the cars within the site during the work day.

In the other two other tunnels, the contractor elected to fill in the floor of the tunnel to the top of rail to allow his operation to utilize rubber-tired equipment such as trucks, loaders and drill jumbos.

In the case of the rail borne operation, rock removed from the roof was generally collected in side dump cars, while in the case of rubber-tired access, fabricated steel blasting shields were placed over the track to protect it from the rock fall.

See Figure 7.

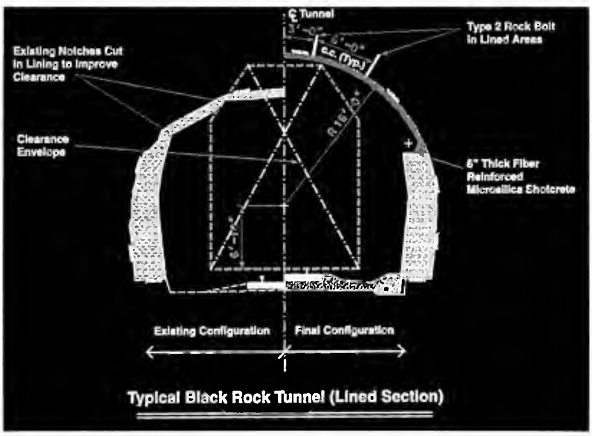


Figure 6

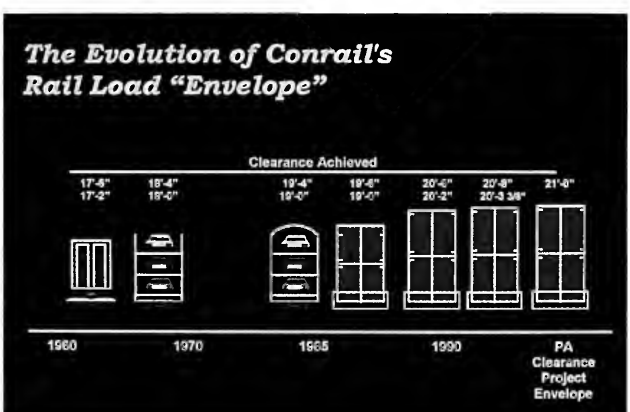


Figure 7

The case of the Allegheny tunnels presented a somewhat different situation. with each of the two locations having multiple tunnels, each carrying a single track as of the start of the project. Spruce Creek, involved a pair of tunnels, one of which was built as a double-track tunnel in 1850, the other built as a double-track tunnel around 1900. Both, long since, had been single-tracked to provide clearance for the increasing engine and car sizes in the first half of this century. The decision was made to enlarge one of the two tunnels to a sufficient size that both tracks could be routed through it with the full 21' clearance, allowing the older tunnel to remain intact to preserve it as a historical site. This decision to enlarge the westbound tunnel for two tracks was soon found to have been a wise decision, when preliminary engineering for enlarging the tunnel established that the original tunnel had been significantly over excavated and then backfilled around the final Ashlar stone and brick lining. Combining this situation with the fact that a 12" to 16" steel fiber reinforced shotcrete lining would structurally replace a 30" masonry lining allowed us to re-establish the double-track tunnel with minimal rock excavation. It was further decided to remove all existing soft material from the floor of the tunnel as well, to reprofile the floor rock in cross section to improve drainage and forestall ballast contamination. Work began on the Spruce Creek Tunnel in late January 1994 after a temporary CP (Control Point) was installed to route the 60 daily trains through the old eastbound tunnel on a TCS (Traffic Control System) single-track gauntlet, until the new enlarged tunnel was placed in service in July 1994

In the case of the Gallitzin Tunnels, Conrail has three separate tunnels:

The first and shortest is the No. 1 Track or Portage Tunnel which was originally built by the Portage Railway as a double-track tunnel. Although this tunnel existed as an unlined rock tunnel from approximately 1850 through 1900, continued freeze/thaw caused minor rock fall and icing problems led to its lining at that time. In the first 60 years of this century, clearances were improved a number of times through a combination of single tracking and lowering the floor as evidenced by several sets of deteriorating underpinning benchwalls.

See Figure 8.

As the three Gallitzin Tunnels had to be progressed sequentially, work began with Portage Tunnel. It was designed in the Summer of 1993 and enlarged between September and December of 1993. Track

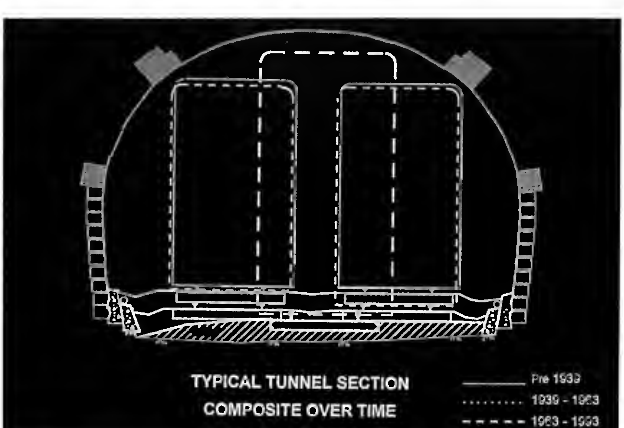


Figure 8

through the tunnel, 1100 feet of track on the east approach and 2800 feet of track on the west approach, was removed by the railroad, and a Contractor was then utilized to reinforce the tunnel, underpin the walls, and lower the floor. The first stage of this work involved structural repairs to portions of the lining which were in distress and installation of a reinforced shotcrete lining of the existing brick lining to stabilize and support that liner during blasting. The side walls were then underpinned by pin piles, drop hammer drilled and socketed in the underlying rock, mobilizing them to allow the rock to be removed from the floor, after which new reinforced benchwalls were constructed.

See Figures 9 and 10.

The Unit prices realized for the underpinning for the No. 1 Track Portage Tunnel caused the estimate of the cost of underpinning both the No. 2 and No. 3 tunnels to increase significantly. As in

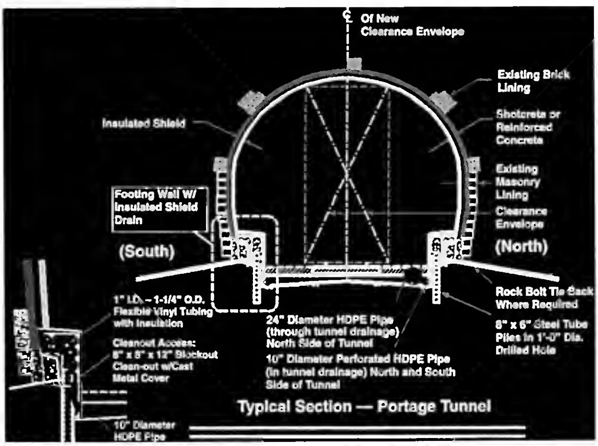


Figure 9

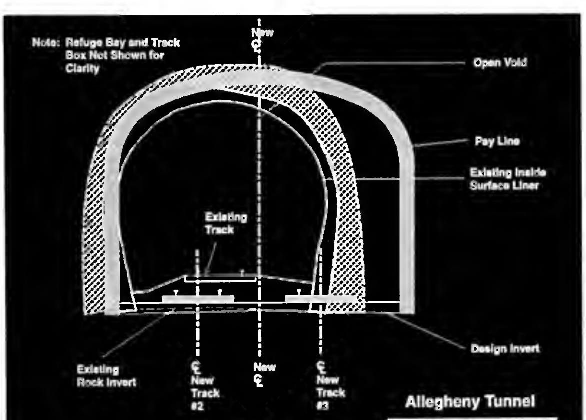


Figure 10

Spruce Creek, the site investigation indicated that the original Allegheny Tunnel in many locations had been significantly over excavated only to be backfilled with masonry lining which sometimes covered large voids. Faced with the significantly high cost estimate to clear two tunnels paralleling each other within a distance of 60', it was concluded that the more economical solution would be to enlarge the better of the two, the Allegheny Tunnel, to restore it to a double-track tunnel. By utilizing the same lining cross section for a double-track Allegheny Tunnel as was used at Spruce Creek, a considerable cost savings through the combination of both of these tunnels in one large contract was realized. In order to expedite tunnel enlargement, all traffic was rerouted through the parallel Gallitzin Tunnel, and work on the Allegheny Tunnel began in July 1994 with headings from both the east and west ends.

See Figure 11.

The Freeport coal seam, approximately four and one-half feet thick and abutted by thin weak shale layers as well as an underclay associated with the shale, is approximately 20' above the roof of the tunnel in the area of the east portal, dipping at an angle of two to three degrees, while the tunnel profile is rising at a one percent grade. The coal seam intersects the tunnel and appears in the new tunnel roof approximately 500' in from the east portal (Sta. 0+00) and continues to drop relative to the tunnel cross section, until at Station 19 the coal is below the tunnel floor. Based upon the research information and mining maps available to us at the time of design, we were advised that the coal mining occurred west of Station 23, an area where any mining was well below the tunnel floor. During preliminary engineering over 60 probe holes were drilled in the existing #2 and #3 Tunnel liners, as well as four full depth rock cores 200' down from the surface just north of the north wall of the Allegheny (#2) Tunnel. Most found the coal and only a few indicated voids in the coal seam. As things turned out, this information was incorrect as a significant amount of coal in the Freeport Seam had been illegally mined near the turn of the century. With the tunnel fully lined by that point, the railroad was unaware that the mining had progressed behind the tunnel liner and elsewhere in the mountain surrounding the tunnel.

On August 2nd, the contractor had just completed a 6' round of excavation at approximately Station 1+30 when a collapse occurred at Station 0+90. This collapse opened up a void in the roof extending 40' above the theoretical excavation line, 30' across the tunnel width and approximately 40'

Figure 11

Figure 12

along the tunnel length. Shortly thereafter, a second collapse occurred which expanded the void through to the surface approximately 80' above the tunnel. The volume of material which collapsed was approximately 1200 tons. Fortunately no one was at the location at the time of the collapse and, therefore, no injuries occurred. Investigation indicated that excavation had broken through 13 feet to the coal above. It is suspected that an abandoned mine access shaft from the surface may have been the cause for the "glory hole," which formed to the ground above.

See Figure 12.

Faced with uncertain strata surrounding the collapse, the Contractor backed off on the east heading to the point where he had installed the steel sets for the initial portal construction and proceeded to support the entire east heading from the portal with steel sets. Work on the east heading progressed very slowly from the collapse west as sets were installed and the heading advanced by 4' increments as each set was installed until we reached Station 2+65. By this time, the contractor had elected to accelerate the set installation procedure by increasing the number of sets placed and blocked before dropping back to concrete the arch. Typically, the contractor had been shooting a four foot round, placing and blocking a set, and then repeating the cycle. After two sets were in place, the arch would be concreted fully. In this instance, the contractor attempted to excavate and block four sets and take a fifth round of 4' of excavation before he concreted the four standing sets.

At 2:30 in the morning of October 27th, the contractor shot a 4' round between Station 2+61 and Station 2+65. 45 minutes later as the equipment was completing the mucking operation, creaking and crushing sounds were heard from the blocking above the sets which had not been concreted. All personnel were withdrawn from the heading just before the roof collapsed. All four free-standing and blocked sets which had been erected but not concreted were crushed and swept aside as approximately 1500 tons of debris came sliding down. Set #65, the last one concreted succeeded in holding the rip in the roof and prevented it from moving any further down the heading. Meanwhile the existing old tunnel arch, which was intact from Station 2+68 west, succeeded in holding the collapse from moving west. Inspection of the collapsed area indicated not only a coal seam in this location where anticipated but it was virtually void of coal on the north side, having been previously mined out and in some places the unsupported mine roof had collapsed into the original coal area.

Figure 13

Correction of this situation involved installing a temporary bulkhead on the east side of the collapse debris and concreting the entire void to establish a structural mass through which we could safely drill, shoot, and excavate. At this point, project management had a major situation to resolve and elected to launch into an in-depth, drilling, and probing plan, which revealed that this mined-out and collapsed mine roof situation could be expected to continue for another 600-700 feet to approximately Station 9. The only viable correction for this situation was to begin a sequence of drill, grout, and concrete operations to fill all of the voids in the coal seam as well as to tie together the collapsed debris into a mass which could be safely rock bolted and mined. Needless to say, production was slow as the grouting progressed and the east heading proceeded forward one set at a time. On March 2, 1995 the last set was installed at Station 3+60 and the Contractor resumed normal two stage split face excavation. Excavation was completed by the end of June 1995 when the headings met. Approach work, drainage, and track construction quickly followed allowing us to reconstruct track in August through the tunnel.

Fittingly, the Allegheny Tunnel was both the largest single piece of our clearance project at $29.6 million and the last on the primary route to be cleared with the first train through the enlarged tunnel on August 28, 1995.

See Figure 13.

Completion of the final five miles of route into the Port of Philadelphia was complicated by modifications of several concrete tunnels carrying multiple 48" water mains beneath the tracks to be lowered, very poor clay layers beneath one end of the Belmont Tunnel carrying Conrail's route to the Port under tracks of Amtrak's Northeast Corridor, and major clearance problems with the catenary in Zoo Interlocking on Amtrak's Northeast Corridor. The only feasible solution to the 20'-2" double-stacks passing through 18'-6" catenary was a total physical separation of both the track and signal system for Conrail's single freight track and the remainder of Amtrak's Zoo Interlocking. Work in the three mile section with three overhead bridges, one cut and cover tunnel, and Zoo Interlocking was completed December 22, 1995 with surfacing of the final section of track.

Proposed 1997 AREA Manual and Portfolio Revisions

The following proposed Revisions of the AREA *Manual for Railway Engineering and Portfolio of Trackwork Plans* have been recommended to the Association by the Technical Committee responsible for each after a letter ballot is approved by: (1) a two-thirds majority of the eligible members voting, and (2) by at least fifty percent of the total eligible voting members on the committee. They are being published here for comment by the general AREA membership and any other interested parties. Comments should be sent to AREA headquarters by February 1, 1997. These comments will be considered by the Board of Direction in deciding whether to give final approval for inclusion of the proposed changes in the Manual and Portfolio Revisions, which if approved, go into effect August 1, 1997.

Proposed 1997 Manual Revisions to Chapter 1—Roadway and Ballast

Part 10—Geosynthetics

Page 1-10-6, Article 10.1.2., Material Requirements. Replace current Table 10-2 on Geotextile Property Requirements for Railroad Track Stabilization with the following revised Table 10-2:

Table 10-2. Physical Property Requirements for Railroad Track Stabilization

Test methods for Geotextiles	**Regular 10–12 Oz/ Sq Yd***	**Heavy 12–16 Oz/ Sq Yd***	**Extra Heavy 16–20 Oz/ Sq Yd***
Grab Tensile Strength - ASTM D4632 lbs (N)	175 (775)	225 (1000)	350 (1555)
Elongation at Failure - ASTM D-4632 (%)	20	20	20
Mullen Burst Strength - ASTM D-3786 psi (kPa)	400 (2750)	450 (3100)	620 (4270)
Planar Water Flow/Transmissivity ASTM D 4716 (Sq ft/min x 10^{-3} @ Normal Stress of 3.5 psi and i = 1.0)	2 (.18)	4 (.37)	6 (.56)
Coefficient of Normal Permeability (K) (cm/sec) ASTM D-4491	0.1	0.1	0.1
Permittivity ASTM D-4491 (Sec.$^{-1}$)	0.30	0.25	0.20
Apparent Opening Size ASTM D-4751 (U.S. Standard Sieve No.) U.S. Standard Sieve Number larger than	70	70	70
Trapezoid Tear Strength - ASTM D-4533 lbs (N)	100 (444)	130 (575)	150 (665)
Puncture Strength - ASTM D-4833 lbs (N)	110 (485)	150 (665)	185 (820)
Abrasion Resistance ASTM D-4886 % strength retained in breaking load	**	**	**

*Mass per unit area: The values indicated for the classification of material are for information only. It is recommended that the selection of material be based on the above recommended index property values shown in these tables. Material selection should not be limited by mass per unit area; i.e., geotextiles may accomplish the same purposes with more or less mass per unit area.

**Abrasion resistance of geotextiles can be evaluated through the use of ASTM D-4886 Test Method For Abrasion Resistance of Geotextiles (sand paper/sliding block method). The abrasion resistance of geotextiles is application specific. The Engineer should evaluate the specific application to determine if abrasion resistance is critical and consult the manufacturer to obtain values and determine the site specific requirements for the application in question.

Proposed 1997 Manual Revisions to Chapter 4—Rail

Part 2—Specifications

Page 4-2-10, Article 2.1.5, Section. Replace current Table 2-4. Section tolerances with following revised Table 2-4. Which includes the addition of special trackwork rail section tolerances and new notes 3 and 4.

Table 2-4. Section Tolerances

	Inches (Thousandths)			
	Rail		Trackwork Rail	
Description	Plus	Minus	Plus	Minus
Height of rail (measured within 1 ft from end)	0.040	0.015	0.040	0.015
Width of rail head (measured within 1 ft from end)	0.030	0.030	0.015	0.015
Thickness of web	0.040	0.020	0.040	0.020
Fishing template standout	0.060	0.000	0.030	0.000
Asymmetry of head with respect to base	0.040	0.040	0.030	0.030
Width of base	0.050	0.050	0.030	0.030
Flange height	0.020	0.015	0.015	0.015

Note 1: Base concavity shall not exceed 0.010 inch. Convexity is not permitted.
Note 2: No variation will be allowed in dimensions affecting the fit of the joint bars, except that the fishing template may standout not to exceed 0.060 inch laterally.
Note 3: All 4 corners of the rail base shall have the radii according to the drawing +/– 1/32". Any disputes shall be analyzed on an Optical Comparator.
Note 4: The section of the rails to be used in AREA trackwork shall conform to the design specified by the purchaser subject to the tolerances listed under trackwork rail above.

Proposed 1997 Manual Revisions to Chapter 8—Concrete Structures and Foundations

Part 1—Materials, Tests and Construction Requirements

Page 8-1-16, Article 1.5.2, (a). Welding. Revise first and second sentences to read:

"(a) Welding of reinforcing bars shall conform to "Structural Welding Code—Reinforcing Steel", ANSI/AWS D 1.4 of the American Welding Society. Type and location of welded splices and other required welding of reinforcing bars shall be indicated on the plans or in the project specifications. The ASTM designations for reinforcing bars. . . ."

Page 8-1-19, Article 1.5.3.2, (a) Coated Reinforcements, Table 1-11. Insert under "Type" and "Designation," after the first entry:

"Epoxy–Coated Prefabricated Reinforcing Steel Bars ASTM A 934"

Page 8-1-20, Article 1.5.4, Bending and Straightening. Designate present paragraph 1.5.4(a). Second sentence should be modified to read:

"Field bending and/or straightening of bars that are partially embedded in concrete shall be done. . ."

Add a new sub-article "(b)":

"(b) Field bending and/or straightening of epoxy–coated reinforcing bars conforming to ASTM A 934 shall be prohibited."

Page 8-1-23, Article 1.7.3, Reinforcement. Designate present paragraph 1.7.3(a). Add a new sub-article "(b)":

"(b) Epoxy–coated reinforcement shall be covered with opaque polyethylene sheeting or other suitable opaque protective material as approved by the Engineer. For stacked bundles, the protective covering shall be draped around the perimeter of the stack. The covering shall be secured adequately, and allow for air circulation around the bars to minimize condensation under the coverings."

Page 8-1-53. Add the following new section on Specialty Concretes:

1.24 SPECIALTY CONCRETES

1.24.1 General

(a) This manual article describes and provides requirements for specialty concretes that may be used in railroad construction. Before any specialty concrete is used, additional investigation of specific and detailed specifications shall be made.

1.24.2 Sulfur Concrete

1.24.2.1 General

(a) Sulfur concrete is a thermoplastic material produced by mixing heated aggregate 177C to 204C (350F to 400F) with modified sulfur cement and fine mineral filler (ambient temperature) to prepare a well-mixed concrete that is maintained within a temperature range of 132C to 141C (270F to 285F) until placed. The ACI Manual of Concrete Practice contains detailed information.

1.24.2.2 Design

(a) Mixture design for sulfur concrete is different from portland cement concrete.

(b) Aggregate for sulfur concrete shall conform with ASTM C 33.

(c) Reinforcement may be with reinforcing steel, epoxy-coated reinforcing steel or with fibers.

1.24.2.3 Handling

(a) The requirements for mixing/transporting equipment are defined by the unique thermoplastic characteristic of sulfur concrete. Sulfur concrete must be maintained in a molten state and continuously monitored to maintain the temperature range of 133C (270F) to 147C (285F). The concrete mixture must be thoroughly mixed so the molten sulfur cement adequately coats the fine and coarse aggregate and mineral filler.

1.24.2.4 Placing

(a) Sulfur concrete can be placed in either wooden or metal forms.

1.24.3 Heavyweight Concrete

1.24.3.1 Design

(a) Heavyweight concrete, unless otherwise stipulated, shall conform to the other requirements of Chapter 8, Part 1, shall be made with Type II cement, and shall be proportioned as directed by the

Engineer, with not more than 22.7 L (6 gal.) of water per 42.8 kg (94 lb.) of cement. Where heavyweight concrete is required for counterweights, the coarse aggregate shall be trap rock, iron ore, or other heavy material or the concrete may incorporate steel punchings or scrap metal. The mortar shall be composed of 1 part of cement and 2 parts of fine aggregate. Fine metallic aggregate shall consist of commercial chilled-iron or steel shot or ground iron, meeting SAE J 444a. All metallic aggregate shall have a specific gravity of 6.50 or greater and be clean and free from foreign coatings of grease, oil, machine shop compounds, zinc chromate, loose scale, and dirt. The maximum weight of heavy concrete shall be 5,050 kg per cu. m (315 lb. per cu. ft.).

1.24.3.2 Placing

(a) Heavyweight concrete shall be placed in layers and consolidated with vibrators or tampers. Heavyweight concrete usually will not "flow" in a form and must be placed uniformly throughout the area and compacted in place with a minimum of vibration. Under no circumstances shall an attempt be made to move heavyweight concrete during consolidation with vibration equipment. Layers shall be limited to a maximum 300 mm (12 inch) thickness. Consolidation shall be by internal vibrators to achieve uniform and optimum density. In heavyweight concrete vibrators have a smaller effective area, or radius of action; therefore greater care shall be exercised to insure that the concrete is properly consolidated. Vibrators shall be inserted at closely spaced intervals and only to a depth sufficient to cause complete intermixing of adjacent layers. Counterweights containing punchings or scrap metal or iron ore aggregates shall be enclosed in steel boxes.

(b) Heavyweight concrete not enclosed in steel boxes shall be adequately reinforced.

1.24.3.3 Determining Weight

(a) For ascertaining the weight of the concrete, test blocks having a volume of not less than 0.1 cu. m (4 cu. ft.) for ordinary concrete, and 0.03 cu. m.(1 cu. ft.) for heavy concrete, and 0.03 cu. m. (1 cu. ft.) for the mortar for heavy concrete, shall be cast at least 30 days before concreting is begun. Two test blocks of each kind shall be provided, and one weighed immediately after casting and the other after it has cured for 28 days.

Page 8-1-61, Commentary. Add the following Commentary on sulfur concrete article:

C1.24.2 COMMENTARY ON SULFUR CONCRETE

C1.24.2.1 General

(a) Sulfur concrete is generally not resistant to alkalis or oxidizers. However sulfur concrete exhibits excellent characteristics of:

1. High strength [in excess of 62 MPa (9,000 psi)] and fatigue resistance;
2. Excellent corrosion resistance against salts and most acids;
3. Extremely rapid set and strength gains and achieves a minimum of 70% to 80% of ultimate compressive strength within 24 hours;
4. Placement year round, above and below freezing temperatures;
5. Very low water permeability.

C1.24.2.3 Handling

(a) Extreme care should be used when handling sulfur concrete to avoid burns.

C1.24.2.4 Placing

(a) Wall construction should be given special consideration to preclude poor consolidation. Preheating the reinforcing steel and forms using infrared or suitable heaters, plus using insulation on the outside of wall forms should be utilized to retain heat during placement.

Part 13—Precast Concrete Box Culverts

Part 16—Reinforced Concrete Box Culverts

Page 8-13-1, Part 13 will be eliminated and combined with new Part 16. Page 8-16-1, current Part 16 combined with current Part 13, revised and republished as a new Part 16, Design and Construction of Reinforced Concrete Box Culverts. The new Part 16, written as a metric (SI) specification with imperial units shown in parenthesis is reproduced below.

American Railway Engineering Association

Part 16M

Design and Construction of Reinforced Concrete Box Culverts

Metric

16.1 GENERAL

16.1.1 Units

(a) The values stated in metric (SI) units are to be used. The imperial values in parentheses are approximate only, and are provided for information only.

(b) Metric ASTM Standards are cited, where available. Corresponding imperial ASTM designations are cited only in the absence of a metric reference.

16.1.2 Definition

(a) A box culvert is a structure which forms one or more rectangular openings through an embankment.

(b) The size designation of a box culvert opening indicates first the width, followed by the height.

16.1.3 Scope

(a) This recommended practice governs the design and construction of one or more opening precast or cast-in-place rigid frame reinforced concrete box culverts on soil foundations.

(b) This recommended practice does not apply to installations where the vertical dimension (H) from the top of the structure to the base of rail is less than 450 mm (18 inches).

(c) This recommended practice does not provide for installation of precast units by jacking. Provisions for jacking must be investigated separately, and will be in addition to the recommendations of this Part.

16.1.4 Notations

		Units	Alternative Imperial Units
b	The width of a box culvert opening.	m	ft
b^1	The horizontal distance between center lines of box culvert walls.	m	ft
h	The height of a box culvert opening.	m	ft

h^{1}	The vertical distance between center lines of box culvert top and bottom slabs.	m	ft
H	The vertical distance between the top of a box culvert and the base of rail.	m	ft
H^{1}	The vertical distance between the center of a box culvert opening and the base of rail	m	ft
I	The impact load applied to the top of a box culvert, as a percentage of W_{LL}.	%	%
I_S	Moment of inertia of the box culvert top slab gross section, per meter (foot) of culvert length.	mm^4	in^4
I_W	Moment of inertia of the box culvert wall gross section, per meter (foot) of culvert length.	mm^4	in^4
k_e	The coefficient of active earth pressure of embankment fill excluding surcharge loading.	none	none
k_S	The coefficient of active earth pressure of embankment fill including surcharge loading.	none	none
K	The ratio of S to R.	none	none
L_d	Lateral live load distribution length illustrated in Figure 16.4.2A.	m	ft
M_A	The maximum negative moment at the exterior corner of a box culvert per meter (foot) of culvert length.	kN.m	kip.ft
M_B	The maximum positive moment in a box culvert top slab near the center of a culvert opening per meter (foot) of culvert length.	kN.m	kip.ft
M_C	The maximum negative moment in the top slab of a box culvert at the top of a center wall per meter (foot) of culvert length.	kN.m	kip.ft
P_e	The uniformly distributed design load on the sides of a box culvert, excluding surcharge loading.	kN/m^2	lbs/ft^2
P_S	The uniformly distributed design load on the sides of a box culvert, including surcharge loading.	kN/m^2	lbs/ft^2
R	The ratio of b^{1} to h^{1}.	none	none
S	The ratio of I_S to I_W.	none	none
V_A	The maximum vertical shear in the top slab of a box culvert, at the face of support near an exterior corner per meter (foot) of culvert length.	kN	lbs
V_C	The maximum vertical shear in the top slab of a box culvert, at the face of support near a center wall per meter (foot) of culvert length.	kN	lbs
W	The total uniformly distributed load on the top of a box culvert; a combination of W_{LL}, W_{DL} , and I.	kN/m^2	lbs/ft^2
W_{DL}	The uniformly distributed dead load on the top of a box culvert.	kN/m^2	lbs/ft^2

W_e	Mass density of embankment fill, taken as 1900 kg/m^3 in metric units (or the weight density of embankment fill taken as 120 lbs/ft^3 in imperial units).	kg/m^3	lbs/ft^3
W_{LL}	The uniformly distributed live load on the top of a box culvert.	kN/m^2	lbs/ft^2
W_S	Mass of concrete per square meter of top slab area in metric units (or the weight of concrete per square foot of top slab area in imperial units).	kg/m^2	lbs/ft^2

16.2 MATERIALS

16.2.1 Existing Foundation Material

(a) The characteristics of existing foundation materials shall be investigated as recommended in Part 22.

16.2.2 Existing Embankment Material

(a) The characteristics of existing embankment materials shall be investigated in conjunction with existing foundation conditions where existing embankment material is to be excavated and reused.

16.2.3 Backfill and Bedding Materials

(a) Backfill and bedding materials shall be subject to the approval of the Engineer. Wet or impervious materials shall not be used except as outlined in Paragraph 16.2.3(g), and all backfill and bedding shall be free from brush and other organic materials.

(b) Crushed stone for bedding shall consist of crushed rock, and shall be graded such that 100% passes a 50 mm (2 inch) sieve, and 100% is retained on a 19 mm (¾ inch) sieve.

(c) Sand for foundation leveling shall consist of selected excavated sand, free from clay and organic materials, and free from rock fragments exceeding 19 mm (¾ inch).

(d) Crushed stone to be placed around drainage pipes shall be the same as for bedding, except that the Engineer may specify a different grading.

(e) Unless otherwise shown on the contract documents, structural granular backfill shall be well graded granular pit run gravel or crushed stone with 100% passing the 106 mm (4¼ inch) sieve and 0% passing the 75μm (Number 200) sieve.

(f) Native or imported backfill materials not meeting the requirements of structural granular backfill may be used subject to the approval of the Engineer.

(g) Clay for seepage barriers shall be clay or silty clay of a medium to high plasticity and of a low permeability, all subject to the approval of the Engineer.

16.2.4 Concrete

(a) Materials for concrete shall meet the requirements of Part 1.

(b) The minimum compressive strength of concrete shall be 30 MPa (4,300 psi) at 28 days.

(c) Concrete materials shall comply with the requirements of Part 1 that affect the durability of the culvert, including alkali-aggregate reactions, sulfate and other chemical reactions, and freezing and thawing. Air entraining and other admixtures shall be used only when approved by the Engineer. Admixtures containing chlorides shall not be used.

16.2.5 Reinforcement

(a) Reinforcing steel shall meet the requirements of ASTM Standard A615M Grade 420 (Grade 60 in imperial units), or ASTM Standard A706, or welded steel wire fabric conforming to ASTM Standard A497, with an allowable tensile stress of 165 MPa (24,000 psi) for service load design.

16.2.6 Miscellaneous Metal

(a) All hardware for sleeves, anchor bolts, inserts and other purposes shall be either hot-dip galvanized in accordance with ASTM Standard A153, or epoxy coated in accordance with ASTM Standard A775M, or stainless steel in accordance with AISI Standard Type 304.

16.2.7 Miscellaneous Materials

(a) Water stops shall meet the requirements of Part 1 for watertight construction joints.

(b) Gasket material shall conform to AASHTO Designation M-198 75 1, Type B, Flexible Plastic Gasket.

16.3 DESIGN METHODS

16.3.1 Design Considerations

(a) The following shall be considered in the design:

1. The purpose for which the structure is to be used.

2. Depth of culvert from base of rail to invert level.

3. Requirements for soil cover above the top of the structure and below the base of rail, as specified by the Engineer in addition to the requirements of these recommendations.

4. Waterway alignment and skew angle.

5. Subgrade width and embankment slopes.

6. Existing foundation conditions.

(b) For precast culverts, the following shall also be considered in the design:

1. Methods to be used for and stresses induced by handling and transportation of units.

2. Method of installation.

16.3.2 Design to Accommodate Flow

(a) The calculation of flow rates and the design of the culvert and approaches to accommodate these flows shall be done in accordance with Part 3 of Chapter 1.

16.3.3 Structural Design

(a) The design shall comply with all provisions of Part 2, except as modified in this Part 16M.

(b) The structure shall be analyzed assuming that all joints between slabs and walls are rigid, with positive and negative bending moments determined by the theory of elasticity.

16.4 DESIGN LOADS

16.4.1 General

(a) The following loads shall be used in the design of box culverts supporting track:

1. dead load,

2. live load,

3. impact load.

(b) The loads, uniformly distributed per square meter (per square foot) to the top of the box, are shown in Figure 16.4.1 for various depths of fill.

16.4.2 Dead Load

(a) The dead load consists of the estimated mass of the track, fill and top slab of the structure, multiplied by the acceleration due to gravity. Dead load shall be uniformly distributed to the culvert as shown on Figures 16.4.2A and 16.4.2B.

(b) The minimum lateral pressure on the sides of the box shall be based on an assumed earth pressure coefficient of 0.33.

(c) The maximum lateral pressure on the sides of the box shall be based on an assumed earth pressure coefficient of 1.0.

(d) As an alternative to Paragraph 16.4.2 (c), the Engineer may at his discretion determine the maximum design mass density of a fully saturated fill, and the corresponding earth pressure coefficient that would apply, and use these in the calculation of both vertical and lateral pressures from dead loads.

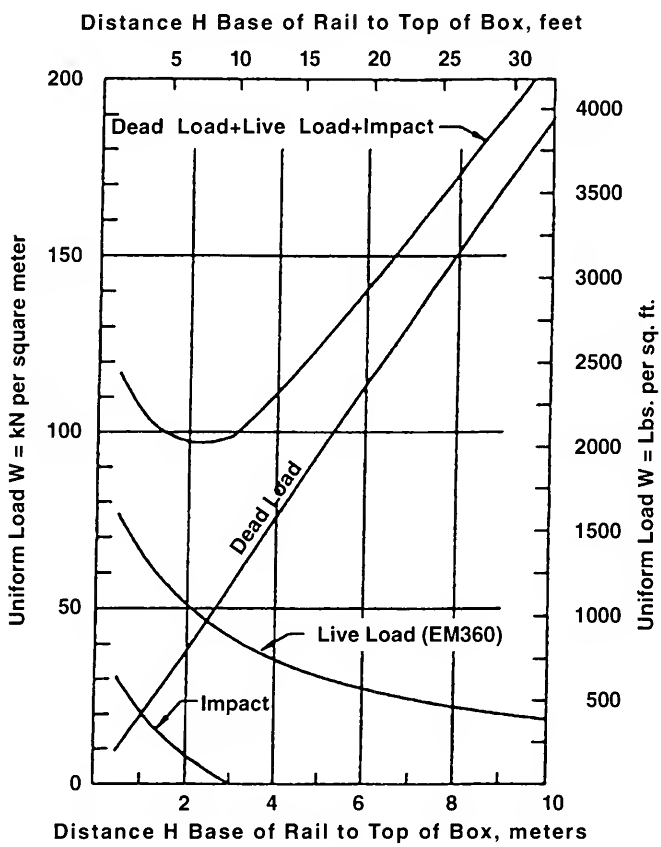
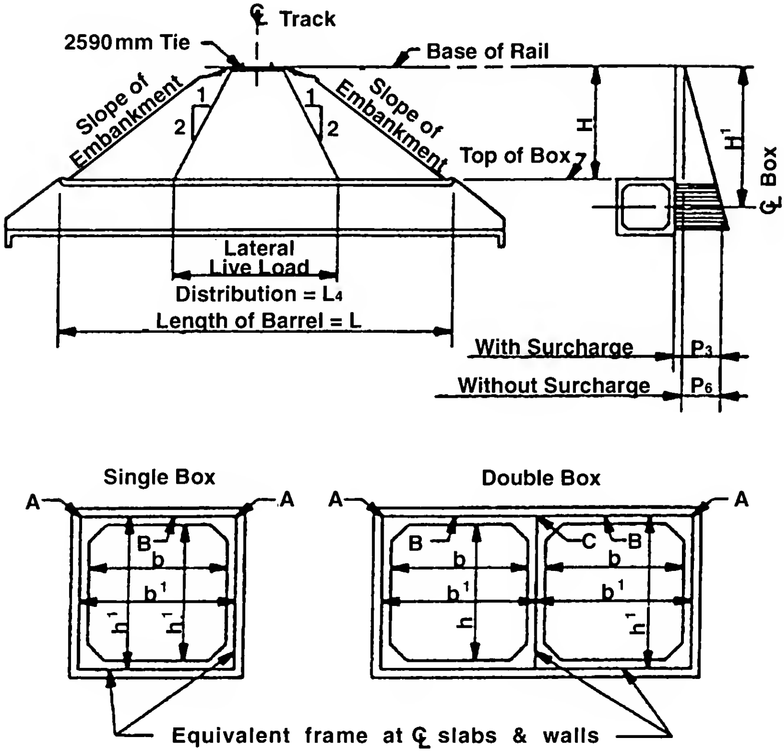


Figure 16.4.1
Uniformly Distributed Load to Top of Box



Notes: 1. Refer also to Article 16.1.3 Notations.

2. b, b', h, h', H and H' are measured in meters in the equations of Figure 16.4.2B, Metric Units, and in feet in the equations of Figure 16.4.2B, Imperial Units.

Figure 16.4.2A

(e) The lateral pressures on each side of the box may be assumed to be uniformly distributed over the entire height, equal and opposite in direction. This assumption has been made in the design equations shown in Figures 16.4.2A and 16.4.2B. If a more exact distribution is used, Figures 16.4.2A and 16.4.2B do not apply.

16.4.3 Live Load

(a) The live load for each track shall be as determined from Part 2. The distribution of the live load to the culvert shall be in accordance with Figures 16.4.2A and 16.4.2B.

(b) No increase in load shall be used for multiple track loadings.

(c) The minimum lateral pressure on the sides of the box shall be calculated using the earth pressure coefficient determined by Paragraph 16.4.2 (b).

(d) The maximum lateral pressures that may be generated on the sides of the box shall be considered in the design, except that the earth pressure coefficient of Paragraph 16.4.2 (c) need not be applied to live loads. If the provisions of Paragraph 16.4.2 (d) are used with respect to dead loads, then they shall be used for the calculation of maximum pressures from live loads also.

Design Constraints Used in Equations

Live Load: E80
Loads on Top Slab:

$$W = W_{LL}\left(1+\frac{I}{100}\right)+W_{DL} = \text{Uniform Load} = \text{psf}$$

$$W_{LL} = \frac{80000}{5\times L_d} = \text{Uniform Load} = \text{psf}$$

$$W_{DL} = W_E H+\frac{200}{L_d}+W_s = \text{Uniform Load} = \text{psf}$$

I (Impact) = From 40% at H = 18 inches
To 0% at H = 10 feet

Loads on Walls:

$$P_e = k_e W_e H' = \text{Uniform Load} = \text{psf}$$

$$P_s = k_s W_e\left(H'+\frac{80000}{5W_e L_d}\right) = \text{Uniform Load} = \text{psf}$$

$k_e = 0.33$ min., 1.0 max.

$k_s = 0.33$

Design Equations for Single Box
(per Foot of Culvert Length)

$$\text{Max. } M_B = \frac{Wb^2}{24}\left(\frac{1+3k}{1+k}\right)-\frac{P_e h^2}{12}\left(\frac{k}{1+k}\right) = \text{lb. ft. use min. value of } P_e$$

$$\text{Max. } M_A = \frac{Wb^2}{12}\left(\frac{1}{1+k}\right)+\frac{P_s h^2}{12}\left(\frac{k}{1+k}\right) = \text{lb. ft. use max. value of } P_e \text{ or } P_s$$

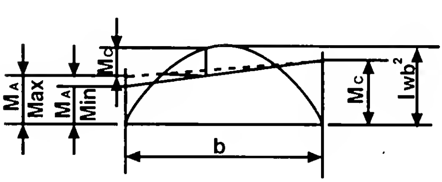
$$V_A = \frac{Wb}{12} = \text{lbs.}$$

Design Equations for Double Box
(per Foot of Culvert Length)

$$\text{Max. } M_A = \frac{Wb^2}{12}\left(\frac{1}{1+2k}\right)+\frac{P_s h^2}{6}\left(\frac{k}{1+2k}\right) = \text{lb. ft. use max. value of } P_e \text{ or } P_s$$

$$\text{Min. } M_A = \frac{Wb^2}{12}\left(\frac{1}{1+2k}\right)+\frac{P_e h^2}{6}\left(\frac{k}{1+2k}\right) = \text{lb. ft. use min. value of } P_e$$

$$\text{Max. } M_C = \frac{Wb^2}{12}\left(\frac{1+3k}{1+2k}\right)-\frac{P_e h^2}{12}\left(\frac{k}{1+2k}\right) = \text{lb. ft. use min. value of } P_e$$



ASSUME M_c SAME FOR M_A MAX. & M_A MIN.

$$V_A = \frac{Wb}{4}\left(\frac{2+3k}{1+2k}\right)+\frac{P_s h^2}{4b}\left(\frac{k}{1+2k}\right) = \text{lbs. use max. value of } P_e \text{ or } P_s$$

$$V_c = \frac{Wb}{4}\left(\frac{2+5k}{1+2k}\right)-\frac{P_e h^2}{4b}\left(\frac{k}{1+2k}\right) = \text{lbs. use min. value of } P_e$$

Figure 16.4.2B
Imperial Units

Design Constraints Used in Equations

Live Load: EM360
Loads on Top Slab:

$$W = W_{LL}\left(1+\frac{1}{100}\right) + W_{DL} = \text{Uniform Load} = \text{kN/m}^2$$

$$W_{LL} = \frac{356}{1.52 \times L_d} = \text{Uniform Load} = \text{kN/m}^2$$

$$W_{DL} = 0.00981\ W_e H + \frac{2.92}{L_d} + 0.00981\ W_s = \text{Uniform Load} = \text{kN/m}^2$$

I (Impact) = From 40% at H = 0.45 m
To 0% at H = 3.0 m

Loads on Walls:

$$P_e = 0.00981 k_e W_e H' = \text{Uniform Load} = \text{kN/m}^2$$

$$P_s = 0.00981 k_s W_e\left(H' + \frac{36,300}{1.52 W_e L_d}\right) = \text{Uniform Load} = \text{kN/m}^2$$

$k_e = 0.33$ min., 1.0 max.

$k_s = 0.33$

Design Equations for Single Box
(per Foot of Culvert Length)

$$\text{Max. } M_B = \frac{Wb^2}{24}\left(\frac{1+3k}{1+k}\right) - \frac{P_e h^2}{12}\left(\frac{k}{1+k}\right) = \text{kN.m use min. value of } P_e$$

$$\text{Max. } M_A = \frac{Wb^2}{12}\left(\frac{1}{1+k}\right) + \frac{P_s h^2}{12}\left(\frac{k}{1+k}\right) = \text{kN.m use max. value of } P_e \text{ or } P_s$$

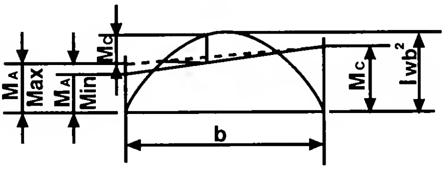
$$V_A = \frac{Wb}{12} = \text{kN}$$

Design Equations for Double Box
(per Foot of Culvert Length)

$$\text{Max. } M_A = \frac{Wb^2}{12}\left(\frac{1}{1+2k}\right) + \frac{P_s h^2}{6}\left(\frac{k}{1+2k}\right) = \text{kN.m use max. value of } P_e \text{ or } P_s$$

$$\text{Min. } M_A = \frac{Wb^2}{12}\left(\frac{1}{1+2k}\right) + \frac{P_e h^2}{6}\left(\frac{k}{1+2k}\right) = \text{kN.m use min. value of } P_e$$

$$\text{Max. } M_C = \frac{Wb^2}{12}\left(\frac{1+3k}{1+2k}\right) - \frac{P_e h^2}{12}\left(\frac{k}{1+2k}\right) = \text{kN.m use min. value of } P_e$$



ASSUME M_C SAME FOR M_A MAX. & M_A MIN.

$$V_A = \frac{Wb}{4}\left(\frac{2+3k}{1+2k}\right) + \frac{P_s h^2}{4b}\left(\frac{k}{1+2k}\right) = \text{ kN use max. value of } P_e \text{ or } P_s$$

$$V_c = \frac{Wb}{4}\left(\frac{2+5k}{1+2k}\right) - \frac{P_e h^2}{4b}\left(\frac{k}{1+2k}\right) = \text{ kN use min. value of } P_e$$

Figure 16.4.2B
Metric Units

16.4.4 Impact Load

(a) Impact load shall be added to the live load as determined from Figures 16.4.2A and 16.4.2B, and shall be distributed to the culvert top slab in the same manner as the live load.

(b) No impact shall be added to the lateral forces on the sides of the box.

16.4.5 Other Forces

(a) Centrifugal force, wind force, and longitudinal forces resulting from starting and stopping of trains need not be considered.

16.5 DETAILS OF DESIGN

16.5.1 General

(a) The contract documents shall show in detail all elements of the construction, including dimensions, spacing and size of reinforcement, permitted locations for the placement of handling devices and holes in the case of precast, construction and expansion joints, water stops, waterproofing, and drainage. The maximum foundation pressure shall also be shown.

(b) When it is anticipated that a large number of culverts will be built, standardization of the design and details is recommended.

(c) The culvert shall be designed with a longitudinal camber, where required by the Engineer to counteract the effects of settlement.

16.5.2 Wingwalls

(a) Wingwalls may be cast-in-place or precast.

(b) Wingwalls shall be of such slope and length as to retain the embankment and maintain the culvert opening.

(c) Wingwalls may be straight or flared as local conditions and hydraulic design require.

16.5.3 Barrel and Apron

(a) The cover of concrete over reinforcement shall be 50 mm (2 inches) unless approved otherwise by the Engineer. This requirement does not apply at the joints of precast units.

(b) The same barrel section shall be used throughout, except under very deep fills where a reduced barrel section may be used toward the ends of the box. Consideration shall be given to the construction of future tracks.

(c) Wall and top and bottom slab thicknesses shall be a minimum of 250 mm (10 inches), or as required by the Engineer. Greater wall and slab thicknesses should be considered for cast-in-place construction to facilitate concrete placement.

(d) Haunches shall be provided, and shall have a minimum leg length equal to the thickness of the top slab.

(e) In long culverts, or culverts under high fills, consideration should be given to the placement of joints to provide for possible vertical and longitudinal movements of the barrel of cast-in-place culverts. If joints are used, the first joint shall be not less than 3 meters (10 feet) from the end of the cast-in-place barrel. For cast-in-place construction, joints should not be placed in regions of maximum load.

(f) Precast units shall be designed with tongue and groove or male and female ends such as shown in Figures 16.5.3A and 16.5.3B or as determined by the Engineer. The inside face reinforcement shall extend into the male portion of the joint, and the outside face reinforcement shall extend into the female portion of the joint.

(g) Where differential deflection from live load between units exceeds b/800, (where b is the width of the box opening) joints between precast units shall be capable of transferring shear loads through the top slab between adjacent units by a method or devices which may be mutually agreed upon by the box culvert manufacturer and the Engineer. If individual shear connectors are used to fasten the adjacent top slabs together, they shall be spaced no more than 750 mm (30 inches) on center, with a minimum of two shear connectors per joint.

(h) The floor of the barrel and apron may be sloped toward the center. Flow energy dissipation may be provided by texturing the floor of the culvert if this is taken into account in the flow capacity design.

(i) The surface of the top slab in contact with the backfill may be sloped toward the sidewalls for drainage.

(j) The length of the apron, and rip-rap requirements, shall be determined by field conditions in accordance with Chapter 1.

(k) Cutoff walls shall be used at inlet and outlet ends to a depth consistent with the field conditions and potential scour.

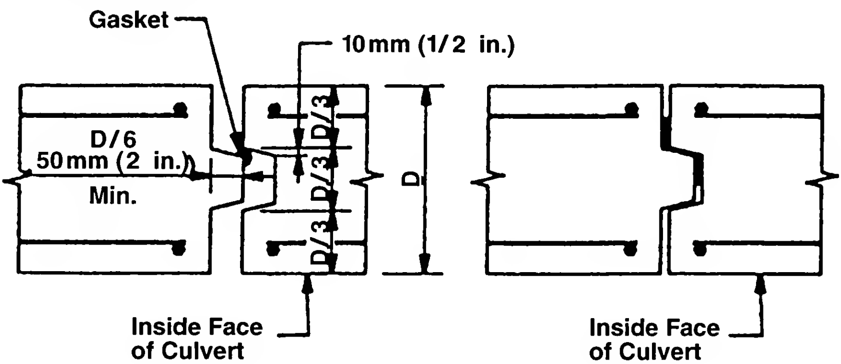


Figure 16.5.3A

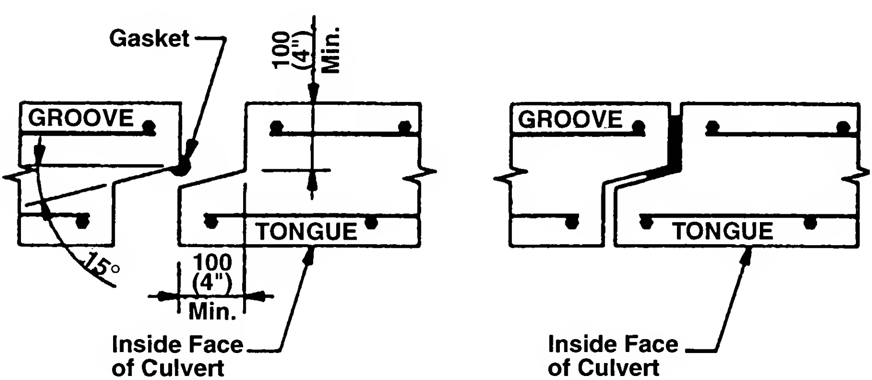


Figure 16.5.3B

16.5.4 Longitudinal Reinforcement

(a) The minimum longitudinal reinforcement in the top slab, bottom slab and walls shall be as follows:

1. 0.4% of concrete cross sectional area for fill depths over the top of the top slab equal to or less than 3 meters (10 feet).

2. For fill depths greater than 3 meters (10 feet), this percentage shall be increased proportionally to 1.0% for fills of 30 meters (100 feet).

(b) The minimum reinforcement determined from Paragraph 16.5.4(a) shall be provided half on each face of the slab or wall.

16.5.5 Drainage and Waterproofing

(a) Pipe drains in the backfill adjacent to the side walls shall be shown on the contract drawings when necessary. Horizontal drain pipes shall be not less than 200 mm (8 inches) in diameter, perforated, and in such a position that they will function properly. Provisions shall be made for cleaning drainage pipes.

(b) Special provision may be made for waterproofing by use of non-corrosive water stops in accordance with Part 1 for watertight construction joints, and/or by use of a waterproofing membrane in accordance with Chapter 29.

16.5.6 Backfill

(a) The extent of structural granular backfill shall be shown on the contract drawings.

(b) Where structural granular backfill is not required, the Engineer shall specify the materials to be used.

(c) When a seepage barrier is required, the details of its location and thickness shall be shown on the contract drawings.

(d) The Engineer shall specify any other backfill details required.

16.6 MANUFACTURE OF PRECAST UNITS

16.6.1 General

(a) Manufacturer's shop drawings shall be submitted to the Engineer for review.

(b) Precast reinforced concrete culvert units shall be manufactured using steel forms and cured in accordance with Part 1.

(c) Concrete shall be placed by the wet cast method when air-entrainment is specified in the contract documents. When air-entrainment is not specified, the precast reinforced concrete culvert units may be manufactured by the dry cast method if approved by the Engineer.

(d) Handling devices or holes shall be provided where shown on the contract drawings. Details of handling devices shall be shown on the shop drawings and shall be subject to the approval of the Engineer, and shall also satisfy the requirements of Article 16.7.4.

16.6.2 Manufacturing Tolerances

(a) Opening Dimensions—The dimensions of the culvert opening shall vary by not more than +/– 1% from the dimensions shown on the drawings. Such variations shall also be such as to satisfy the requirements of Paragraph 16.6.3(a). The haunch dimensions shall vary by not more than 10 mm (3/8 inch) from the dimensions shown on the drawings.

(b) Slab and Wall Thickness—The slab and wall thickness shall not be less than 95% of that shown on the drawings. A thickness more than that shown on the drawings shall not be cause for rejection.

(c) Length of Opposite Surfaces—Variations in laying lengths of two opposite surfaces of the box unit shall not be more than 10 mm per meter (1/8 inch per foot) of span, with a maximum of 15 mm (1/2 inch) in any box unit, except where bevelled ends for laying on curves are specified on the drawings.

(d) Length of Precast Unit—The length of a precast unit shall vary by not more than 10 mm per meter (1/8 inch per foot) of length from that shown on the drawings with a maximum variation of 15 mm (1/2 inch) in any box unit.

(e) Position of reinforcement—The maximum variation in the position of reinforcement shall be 10 mm (3/8 inch) from that shown on the drawings. In no case, however, shall the as-manufactured cover over the reinforcement be less than 40 mm (1 1/2 inch) as measured to the internal surface or the external surface of the completed box unit unless approved otherwise by the Engineer. This minimum cover limitation does not apply at the mating surfaces of joints.

(f) Area of Reinforcement—The areas of steel reinforcement shall be as required by the drawings. Steel areas greater than those required shall not be cause for rejection.

16.6.3 Physical Requirements

(a) The ends of the units shall be produced with joints as shown on the contract drawings, and so formed that when the units are laid together they will make a continuous line of box units with a smooth interior free of irregularities exceeding 10 mm (3/8 inch) at the joints.

(b) The manufacturer may use alternate joint details to those shown on the contract drawings subject to the approval of the Engineer.

(c) When concrete is placed by the wet cast method concrete compressive strength shall be determined from cylindrical concrete specimens made in conformance with ASTM Standard C39, and prepared in conformance with ASTM Standard C31.

(d) When units are manufactured by the dry cast method, cylinders shall be made in conformance with ASTM Standard C361M, Article 10.3.2.

(e) At least five test cylinders shall be prepared from each day's production of concrete.

(f) Compression test requirements shall be in accordance with ASTM Standard C361M, Article 10.3.3.

16.6.4 Marking

(a) The following information shall be clearly marked on each box unit by indentation, waterproof paint, or other approved means:

1. Project name.
2. Date of manufacture.
3. Name or trademark of the manufacturer.
4. Identification of the plant.
5. Location number/match mark.
6. Identification of top slab.
7. Mass (weight) of unit.

16.7 CONSTRUCTION

16.7.1 Construction Tolerances

(a) The construction tolerances of Paragraphs 16.6.2 (a), (b), (e), and (f) shall also apply for cast-in-place concrete.

16.7.2 Joints

(a) Joints shall be located as shown on the contract drawings or as approved by the Engineer. Joints in cast-in-place box culverts shall be formed as prescribed in Part 1.

(b) Premolded bituminous filler at least 12 mm (½ inch) thick may be used at joints in cast-in-place box culverts.

(c) Precast units shall be placed against previously completed units in such a manner as to assure an adequate seal.

16.7.3 Waterproofing or Dampproofing

(a) Waterproofing, if any, shall be provided as specified by the Engineer.

(b) Where no waterproofing is specified, the surface in contact with the backfill shall be dampproofed in accordance with the provisions of Chapter 29.

16.7.4 Handling Devices

(a) Following installation of precast units, and before waterproofing or backfilling, all protruding handling devices shall be removed, and all holes and pockets shall be filled with a non-shrink grout approved by the Engineer.

16.7.5 Foundations

(a) The foundation requirements apply where the reinforced concrete box culvert is to be constructed by open cut.

(b) Foundation conditions shall be inspected and approved by the Engineer.

(c) Existing unsuitable foundation materials shall be excavated and replaced with new material as required by the Engineer.

(d) A compacted crushed stone bed shall be provided under precast reinforced concrete box culverts. The depth of the crushed stone bed shall be a minimum of 300 mm (12 inches), and shall extend 300 mm (12 inches) on each side of the precast reinforced concrete box culvert with a minimum one to one side slope as shown on Figure 16.7.5.

(e) In cast-in-place construction, the crushed stone bed may be deleted if foundation conditions are favorable, as determined by the Engineer.

(f) The foundation surface upon which the reinforced concrete box culvert is to be supported shall be carefully graded to the required line and level. A well compacted sand layer not exceeding 100 mm (4 inches) in thickness may be provided directly under a precast culvert, and on top of the crushed stone bedding, to facilitate this.

16.7.6 Backfilling

(a) The backfilling requirements apply where the reinforced concrete box culvert is to be constructed by open cut.

(b) Structural granular backfill shall be used for the entire backfill area unless shown otherwise on the contract drawings, and except as required for:

1. foundations, as recommended in Article 16.7.5; and
2. drainage materials, as recommended in Paragraph 16.2.3(d); and
3. parallel installations as recommended by Paragraph 16.7.6(c).

(c) When precast or cast-in-place reinforced concrete culverts are used in parallel for multicell installations, positive means of ensuring lateral support shall be provided by grouting with non-shrink

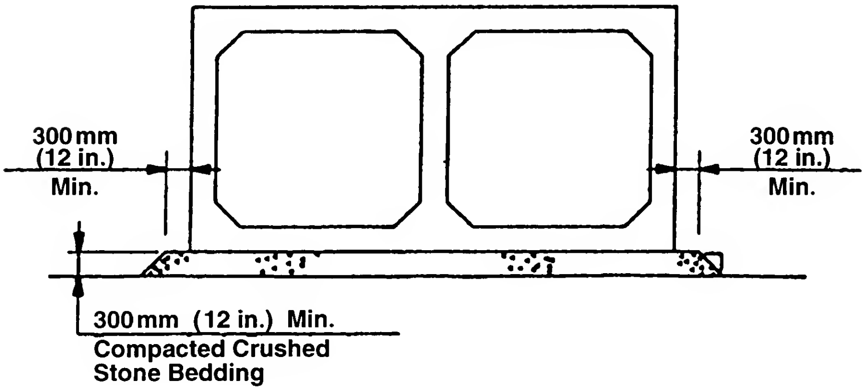


Figure 16.7.5

grout between the units or by filling the space between adjacent units with compacted granular or cementitious material as approved by the Engineer.

(d) Backfill shall be placed alternately on each side of the box and deposited in layers not more than 300 mm (12 inches) thick. The layers shall be horizontal or sloping away from the structure, with each layer carefully tamped.

(e) Care shall be taken in selecting and placing the backfill to prevent damage when the exterior of the culvert has a waterproofing coating or membrane. Protective cover material may be used to prevent damage to the waterproofing system.

AMERICAN RAILWAY ENGINEERING ASSOCIATION

Part 16M Commentary

Reinforced Concrete Box Culverts

C16.1 GENERAL

C16.1.2 Definition

Box culverts are used principally for waterways, but may also be used as pedestrian or livestock underpasses, or for other purposes.

C16.1.3 Scope

The design and construction of reinforced concrete box culverts having more than two openings may be modeled upon these recommendations, but the design equations of Figure 16.4.2B will not apply.

The design of box culverts on pile or rock foundations is governed by support conditions, and box culverts on pile foundations will require a special analysis because of these different support conditions. However, the design of box culverts on rock foundations may be based on these recommendations if the Engineer ensures that there is a sufficiently elastic backfill bedding between the culvert and the rock.

The design and construction of reinforced concrete box structures having a vertical dimension from the top of the structure to the base of rail of less than 450 mm (18 inches) may be modeled upon these recommendations, but the effects of impact loading will require special determination. The design equations of Figure 16.4.2B will not apply, particularly with regard to impact.

Reinforced concrete box culvert installations will normally be by open cut, and the reference to jacking in Paragraph 16.1.3(c) will not apply.

C16.2 MATERIALS

C16.2.4 Concrete

Air entrainment should always be provided where concrete will be subjected to freeze-thaw cycles. To increase the imperviousness of the concrete, air entrainment should also be considered in chemically aggressive environments including dissolved sulfates, industrial effluent, and acid rain. Since the dry cast method is not compatible with air entrainment, the Engineer should consider this when preparing the contract specifications.

The preparation of cylinders for determining concrete compressive strength differs for wet cast and dry cast concrete. The Engineer should determine the methods employed by potential manufacturers when preparing the contract specifications.

C16.3 DESIGN METHODS

C16.3.3 Structural Design

A box culvert may be designed as a rigid "U" shape, with a top slab acting as a simple span without negative corner moments. A box culvert may also be designed as an "inverted 'U'", cast or placed upon a separate footing slab. Design of such culverts may be modeled upon these recommendations but the design equations of Figure 16.4.2B will not apply.

C16.4 DESIGN LOADS

C16.4.2 Dead Load, and
C16.4.3 Live Load

Pressures applied to a box culvert will vary with soil moisture content, and over time with increased compaction under traffic. To accurately account for these changes, it would be necessary to determine a range of soil mass density, earth pressure coefficients, and hydrostatic conditions. These would then be applied in combinations to determine both maximum positive and maximum negative moments.

Articles 16.4.2 and 16.4.3 permit such an approach, but also offer a simplified method. The intent of Paragraph 16.4.2 (c) with regard to a maximum design earth pressure coefficient for the application of dead loads is to approximate the more rigorous analysis of maximum negative moments.

C16.5 DETAILS OF DESIGN

C16.5.5 Drainage and Waterproofing

Waterproofing will not normally be required for reinforced concrete box culverts. However, the Engineer may require waterproofing at special installations, such as where culverts are to serve as pedestrian underpasses.

C16.7 CONSTRUCTION

C16.7.5 Foundations

The Engineer may determine that special foundation requirements should apply, for example where precast culverts are to serve as pedestrian underpasses. In such cases, grillage supports may be considered in order to control differential displacements.

Proposed 1997 Manual Revisions to Chapter 14—Yards and Terminals

Part 2—Freight Yards and Freight Terminals

Page 14-2-26, Article 2.5.2., Gradients.

Amend text under section 2.5.2.4.C quote ". . . , unless tracks are equipped with one of the recently developed continuous control devices inside the clearance point and/or on a portion of the track or a tangent point retarder."

To read as ". . . , unless the ladder lead tracks are equipped with one of the multi-unit distributive type of the retarder systems as discussed in section 2.5.4.

Page 14-2-27. Add the following new article:

2.5.4 Ladder Track Yards with Car Speed Control

2.5.4.1 Introduction

When designing, what has traditionally been known as a flat yard, it is not possible to select a gradient for the ladder lead tracks that is compatible with the rollability values of all cars. If the gradient selected is suitable for the average rollability car, then those with a low rollability coefficient will accelerate to unacceptable speeds and conversely, those with a high rollability coefficient may stall on the track before reaching their switch destination.

2.5.4.2 Car Speed Control on Ladder Lead Tracks

A method of overcoming this problem is to introduce the multi-unit type of hydraulic retarders and distribute them throughout the length of ladder lead tracks to provide continuous car speed control. It is then possible to select a gradient that has sufficient inclination to ensure that the high rollability coefficient cars will reach the farthest clearance point in the classification tracks, but at the same time, the retarders will control the acceleration of the low rollability coefficient cars and limit their velocity to a predetermined maximum.

For yards constructed in warm and temperate climates, typical ladder lead track gradients can be in the order of 0.4% to 0.45%; and for locations where low temperature conditions are experienced typical gradients can be 0.5% to 0.75%. If possible, the lengths of the ladder tracks should be restricted to around 1,000 feet from the king switch to the last switch to minimize stalled cars during inclement weather conditions. These typical parameters are based upon a car velocity of approximately 6.0 mph; if higher speeds are selected then the gradients could be less, or the length of the ladder lead tracks extended; if lower speeds are chosen then the inverse applies. The use of tandem turnouts can limit the length of the ladder lead tracks and provide for about 32 classification tracks.

2.5.4.3 The Addition of a Mini-Hump

The efficiency of the switching operation can be enhanced by constructing a mini-hump on the switching lead track. This hump would assist the uncoupling procedure and enable a continuous humping process, replacing the normal flat yard 'drilling' method of operation. A hump profile of around 200' x 1.0% could be used to accelerate all cars to 6.0 mph retarder control velocity on the ladder lead tracks; if higher speeds are required then the hump elevation can be increased to give additional potential head. A number of retarders would be needed on the hump to control the acceleration of the minimum rollability cars.

2.5.4.4 Additional Coupling Speed Control

If a maximum allowable coupling speed is an operational requirement in the classification tracks then this can be achieved by extending the retarder system to include these tracks. Retarders can be located at the tangent points to decelerate the cars to the specified coupling speed. To further enhance both the coupling speed control and the car penetration down the classification tracks suitable gradients can be constructed with retarders distributed along the tracks to form continuous speed control sections. There are many combinations of track gradients and length of speed control sections that can be employed; the final solution would be dependent upon the degree of coupling speed control that is specified.

2.5.4.5 Diagram 1

A Ladder Track Yard with Car Speed Control is illustrated in Diagram 1. All values are typical only, but the plan and profiles are based upon an actual design for a yard located in a warm climate area; the parameters for that design included:

a) Maximum car weight = 270,000 lbs.

b) Rollability ratio = 2 lbs/ton minimum to 8 lbs/ton maximum

c) Coupling speeds = 4 mph average & 6 mph maximum

A Mini-hump was added to enhance the switching operation and to accelerate the cars to 6 mph at the King Switch. The 650' x 0.25% gradient in the classification tracks was included to assist car penetration and the 400' x 0.35% reverse gradient was constructed at the trim end to prevent car run-out.

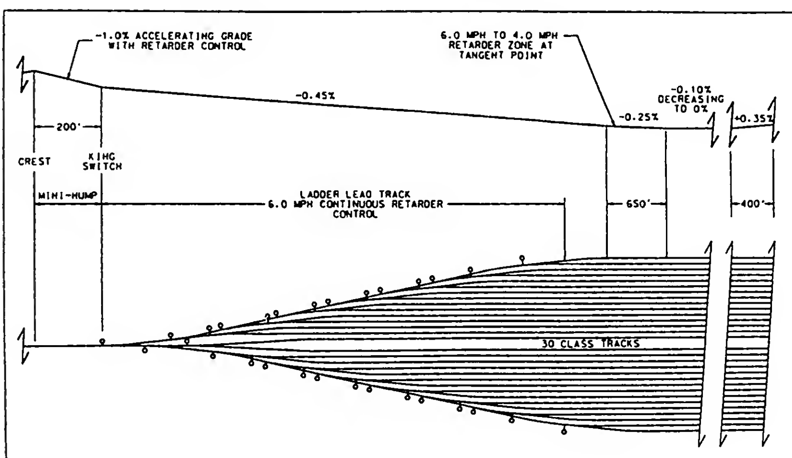


Diagram 1
Typical Track Diagram & Gradient Profile Ladder Track Yard with Car Speed Control

Proposed 1997 Manual Revisions to Chapter 15—Steel Structures

Part 1—Design

Page 15-1-40, Article 1.5.9. Connections and Splices. Replace the current paragraph a.(1) and (2) with the following revised wording:

1.5.9 Connections and Splices

a. Connections and splices, except as used in paragraph d below for milled splices in compression, shall be in accordance with the following provisions:

(1)a. Splices of main members shall have a strength not less than the capacity of the member and shall satisfy the requirements of Art. 1.7.5 and 1.7.6.

b. End connections of main members receiving load from the combined effect of floor system and truss action shall have a strength not less than the capacity of the member. End connections of members carrying direct load from one floorbeam only shall be proportioned for at least 1.25 times their computed reactions.

c. End connections of simply supported floorbeams, stringers, and other beams and girders acting and framed similarly, shall be proportioned for at least 1.25 times their computed shear. Alternatively, these connections shall be proportioned for the combined effect of moment and shear.

(2) Secondary and bracing members shall have... (this is an editorial change, the rest to remain as is).

Page 15-1-59. Replace the current Section 1.14 with the following new material:

Section 1.14 Fracture Critical Members

1.14.1 Scope (1997)

Fracture Critical Members and member components (FCMs) have special requirements for materials, fabrication, welding, inspection and testing. The provisions of Section 12, AWS D1.5 "Fracture Control Plan" (FCP), shall apply to FCMs, except as modified herein.

1.14.2 Definitions (1997)

a. Fracture Critical Members or member components (FCM's) are defined as those tension members or tension components of members whose failure would be expected to result in collapse of the bridge or inability of the bridge to perform its design function.

b. Tension components of steel bridges include all portions of tension members and those portions of flexural members subjected to tension stress. Any attachment having a length in the direction of the tension stress greater than 4 inches that is welded to a tension component of a FCM shall be considered part of the tension component and, therefore, shall be considered Fracture Critical.

1.14.3 Design and Review Responsibilities (1997)

a. The Engineer is responsible: for the suitability of the design of the railway bridge; for the selection of the proper materials; for choosing adequate details; for designating appropriate weld requirements; and for reviewing shop drawings and erection plans to determine conformance with the contract documents.

b. The Engineer is also responsible: for determining which, if any, bridge members or member components are in the FCM category; for evaluating each bridge design to determine the location of any FCM's that may exist; for the clear delineation on the contract plans of the location of all FCM's; for reviewing shop drawings to determine that they correctly show the location and extent of FCM's; and for verifying that the Fracture Control Plan is properly implemented in compliance with contract documents at all stages of fabrication and erection.

c. Welding procedure specifications are considered an integral part of shop drawings and shall be reviewed for each contract.

1.14.4 Special Welding Requirements (1997)

The Submerged Arc Welding (SAW) process shall be used for flange and web butt splices, flange to web welds, and box member corner welds unless otherwise authorized by the Engineer.

1.14.5 Notch Toughness of Steel in Fracture Critical Members (1997)

Charpy V-notch (CVN) impact test requirements for steels in FCM's shall be as given in Table 1-15 except as shown in Note 6.

1.14.6 Fatigue (1997)

The stress range shall not exceed the allowable fatigue stress range, S_{Rfat}, listed in Table 1-21 for the appropriate number of stress cycles N and stress category as defined in Table 1-7 and Table 1-9, respectively.

Table 1-21. Allowable fatigue Stress Range S_{Rfat} (ksi) for Fracture Critical Members (Note 3)

(No Change to Table)

Part 3—Fabrication

Page 15-3-18. Revise Article 3.3.5 to read as follows:

"Welds shall be made only by welders, welding operators and tack welders currently qualified, in accordance with AWS D1.5, to perform the type of work required."

Part 6—Movable Bridges

Page 15-6-6, Article 6.1.5. Machinery and Hydraulic Design. Revise the last paragraph as follows:

b. Where operation is by electric motor, these calculations shall consider the speed-torque characteristics of the system to be provided. The rated full-load torque and maximum starting torque of the motor, including the effect of its control system, shall be considered. The overload relay setting shall be provided for operation of the span under Conditions A, B and C of Art. 6.3.6. The speed-torque curves shall be shown on the drawings.

Page 15-6-7, Article 6.1.10. Communication. Revise the first line as follows:

a. Telephones shall be provided. . . .

Revise the last sentence as follows:

. . . . A ringing system shall be provided at the permanent stations where specified by the Company.

Add a new paragraph (b) as follows:

b. As an alternative to a telephone system, three sets of two-way radios may be provided. The sets shall be capable of operating satisfactorily from all locations outlined in a above. The sets shall be the same as used by the Company for railroad operations, but shall operate at a different frequency.

Page 15-6-13, Article 6.2.1. Material. After Babbitt Metal in b add the following two additional materials:

Bolts . . . . A307 Grade A
High Strength Bolts A449

Page 15-6-13, Article 6.2.2. Types of Bridges. Delete b. 1 through 5

Reletter c. to b.

Page 15-6-14, Article 6.2.3. Counterweights, Revise a. starting in the middle of the third line as follows:

. . . . For vertical lift bridges having a vertical movement exceeding 40 ft. the counterweight ropes preferably shall be balanced by auxiliary counterweights or other devices unless specified. Rope unbalance shall be considered when sizing the operating machinery.

Page 15-6-15, Article 6.2.9. Houses for Operators, etc. Add to c. the following:

. . . . . on the waterway. If such positioning is not practical, the system shall provide for closed-circuit television or other means to visually monitor these operations.

Page 15-6-28, Article 6.4.3. Bearing. Add the following after "Wedges" and before "Acme screws" in a.(1):

Wedges, cast steel or structural steel on ASTMB22,
Copper Alloy UNS No. C86300 bronze 1,500

Page 15-6-33, Article 6.5.3. Air Buffers. Add two paragraphs:

e. As an alternative to air buffers, industrial type shock absorbers may be provided, if specified by the Engineer in the contract documents.

f. Air buffers or shock absorbers may be omitted if the control system is designed to seat the span smoothly at a slow speed which will not create undue impact.

Page 15-6-36, Article 6.5.11. Anti-Friction Bearings. Add the following sentence to the end of b.:

. . . . . .shall be provided. Positive means of aligning the cap to the base shall be provided when loads are in a direction other than directly into the base.

Page 15-6-37, Article 6.5.13. Lubrication. Revise the first two lines of e. as follows:

e. Grooves for counterweight sheave sleeve bearings may be in accordance with the requirements of the foregoing paragraph but preferably shall be spiral grooves cut in the lining and served. . . . .

Page 15-6-39, Article 6.5.17. Collars. In the 2nd line, change "cone points" to "dog points"

Page 15-6-44, Article 6.5.26. Set Screws. In the second line, change "cone points" to "dog points"

Page 15-6-49, Article 6.5.36.5. Counterweights. Change the title from "Counterweights" to "Counterweight Pockets"

Page 15-6-73, Article 6.7.4.2. Engines. Revise a. to read as follows:

a. These requirements apply to separately mounted engines and to engines forming part of an engine-generator set (see Art. 6.7.5.12 for generators). Internal combustion engines shall be of the truck or marine type and of the most substantial kind. The engines shall operate at a speed of not more than 2200 rpm but preferably not more than 2000 rpm unless a higher speed is recommended by the manufacturer, and shall be equipped with a governor to limit the maximum speed to the designated value. Unless otherwise specified, the engine shall have not less than 4 cylinders. The engines shall be tested by the manufacturer at his plant to demonstrate that they will develop the rated torque as defined in Art. 6.7.4.1. where used for span operation.

Page 15-6-83, Article 6.7.5.18. Speed Control for Span Driving Motors. Add a new paragraph b. as follows:

b. Solid state variable speed drives for control of AC or DC span driving motors shall provide for smooth, stepless speed control over a speed range of at least 10 to 1. Speed regulation shall be 2 percent or better up to rated motor speed. A closed-loop feedback type speed control system with over-speed detection shall be used. Speed and torque control shall be the full four-quadrant regenerative, with static (contactorless) reversing. Dynamic braking may be utilized as a supplement to regeneration, but shall not be the primary means of controlling overhauling loads. Acceleration and deceleration ramping shall be field adjustable from 2 to 20 seconds. A minimum of two adjustable speed settings shall be provided, one covering a range of approximately 50% to 100%, and one covering a range of approximately 5% to 25% of rated speed. Two independent adjustable settings of torque limiting shall be provided, each covering the minimum range of 50% to 150% rated motor full load torque. Automatic drive shutdown, with fault indication, shall be provided for loss of feedback signal. Each variable speed drive shall be provided with a disconnect circuit breaker and an isolation contactor mounted in the drive cabinet to remove power from the solid state switching components when the span driving motors are not being operated. The solid state speed control shall be a standard product of the manufacturer.

Page 15-6-83, Article 6.7.5.19. Full Magnetic Control and Article 6.7.5.20. Semi-magnetic Control. Combine these two Articles into a new Article 6.7.5.19 Magnetic Control as follows:

6.7.5.19 Magnetic Control

a. The following features shall apply to full magnetic control with master switch:

(1) Master switches

Master switches for the span driving motors shall be cam operated reversing switches with a single handle, and provided with necessary contacts and contact fingers for operating the magnetic contactors. Contacts and wearing parts shall be easily removable and replaceable. The controller shall provide for speed control of the motors.

(2) Parallel or series-parallel operation

For parallel operation for alternating current, and for constant potential parallel or series-parallel operation for direct current, there shall be separate reversing contactors and separate resistors for each motor. Where two motors are connected to one hoisting machine, accelerating contactors shall be common to both motors, unless otherwise specified by the Company. For three-phase alternating current, each phase shall have its own resistors, so designed as to give balanced current in all three phases. Certain of the acceleration contactors shall be controlled by acceleration relays, such that the specified torques in Art 6.7.5.5 are not exceeded. Where common accelerating contactors are not used, the acceleration contactors shall be so designed, or electrically or mechanically connected, that corresponding circuits in each motor control will be made simultaneously, and that in the event one motor is cut out, the control for the motor in service will operate satisfactorily.

(3) Acceleration relays

Adjusting plugs, screws, and nuts, including time limit adjustments, shall be easily accessible to allow for adjustment of relays to the proper timing intervals between acceleration steps. The contacts shall be removable without disturbing the setting of the relays.

(4) Reversing of motors

Magnetic shunt type contactors for reversing the motors shall be installed with a forward and a reverse pole for each motor conductor.

b. The following features shall apply to semi-magnetic control:

(1) For semi-magnetic control, a drum type master switch shall be provided for reversing the motors by contactors controlled by contacts on the master switch. The master switch accelerating contacts shall carry the secondary current at the step applied without exceeding a 30 C temperature rise, and when the motors are operating a full load torque, or at stalled torque if it is less. Reversing contactors, and accelerating contactors used in conjunction with the accelerating contacts of the master switch, shall meet the requirements of Art. 6.7.5.27.

(2) For control of motors in parallel the switches shall be interconnected so that all switches will be operated simultaneously by one handle. The controllers shall be so arranged that the operation of one motor may be cut out without affecting the operation of any other motor.

Page 15-6-84. Insert new Article 6.7.5.20 as follows:

6.7.5.20 Programmable Logic Controllers

a. Programmable logic controllers (PLC) may be used for the sequential control of bridge operations. The PLC's shall be manufactured and tested in accordance with applicable IEEE and NEMA standards. The PLC's shall be installed and grounded in accordance with the manufacturer's recommendations and the requirements of NEC.

b. The following features shall apply to bridge control with programmable logic controllers:

(1) Cold Backup PLC.

Cold backup shall generally be the preferred backup method. For cold backup, two identical PLC's (CPU's only) shall be provided, and shall be wired in place. One PLC shall normally be de-energized and electrically isolated from power source and input/output (I/O) racks via transfer switches or relays until selected for operation. Separate, dedicated power supplies shall be provided for each PLC. Common I/O modules and racks shall be shared by both PLC's, but only electrically connected to the active one.

(2) Hot Backup PLC.

In certain situations, where momentary interruption to the PLC system cannot be tolerated, hot backup may be utilized. For hot backup, two identical PLC's shall be connected so as to be operating simultaneously, with PLC processor error and fault checking, memory and register updating.

(3) Power Conditioning.

All PLC's and I/O racks shall be protected against power source surge and noise problems by the use of a power conditioning system, including surge suppression, in the power line ahead of all power supplies and ahead of all power connections to I/O modules and any other devices, connected to the PLC's. Consideration shall also be given to the use of surge suppression terminal blocks for all conductors connecting to PLC inputs.

(4) System De-energizing.

The PLC system shall be provided with a master control power switch on the control console which directly interrupts all power feeds to I/O modules when control power is turned off. A standby mode may be utilized with such switch in which input modules remain energized.

(5) Emergency Stop.

A maintained-contact Emergency Stop pushbutton shall be provided which interrupts the PLC logic sequence, and simultaneously and immediately directly interrupts all output module power feeds associated with all bridge operating machinery and all other bridge-related moving equipment including roadway gates and barriers if present.

c. A PLC programming terminal shall be furnished with each PLC system. The PLC programming terminal shall be a compact, portable computer with all necessary PLC programming software, hardware, and communications link cables and adapters specific to the PLC installed. All software registrations and product warranties shall be in the Company's name.

Page 15-6-84, Article 6.7.5.22. Switches for Limiting Travel and Speed. Revise a. to read as follows:

a. Limit Switches which will stop the motors and set the brakes automatically at the end of travel shall be provided for the span lock, rail lock, end wedge and wedge motors. The term limit switch includes all types of mechanical switches as well as encoders, resolvers and proximity switches.

Page 15-6-89, Article 6.7.5.38. Electric Wires and Cables. Revise the last line of b. to read as follows:

Resisting 75 C; or Types RHW, USE or XHHW.

Revise c. and d. to read as follows:

c. Insulated wire for connections on the backs of control panels and in control consoles shall be insulated wire conforming to the Underwriters Laboratories requirements for Type SIS or THWN Wire, 600v.

d. Insulated wires for connections to motor resistance grids shall be high temperature appliance or motor lead wire rated 250 C, 600 volts, Types TFE, TGGT, or TKGT. High temperature wires preferably shall be connected to the general purpose type wires within approximately but not less than five feet, and they shall be run between this connection and the resistors in separate conduits.

Page 15-6-92, Article 6.7.5.45. Navigation Lights. Revise the third line in b. by adding:

"aluminum or fiberglass" after "bronze".

Page 15-6-93, Article 6.7.5.48. Spare parts. Add the following two sentences:

j. One complete set of replacement solid state power modules and one replacement circuit board of each type used for each size and type of solid state variable speed drive.

k. One input and one output module of each type installed, and one spare resolver and encoder of each type installed.

Page 15-6-94, Article 6.7.7. Air Brakes. Replace the words "of boiler plate with riveted joints, or shall have boiler plate sides welded to pressure vessel" in b. by "of steel plate sides welded to pressure vessel."

Page 15-B-1. Add the following four items to the Bibliography:

Standard Specifications for Movable Highway Bridges, American Association of State Highway Officials 1938, 1953, 1970; American Association of State Highway and Transportation Officials 1978 and 1988.

Standard Specifications for Movable Bridges, Canadian Engineering Standards Association A20, 1927.

Specification for Movable Bridges (Second Edition) Canadian Standards Association S20, 1960.

Bridge Inspector's Manual for Movable Bridges, U.S. Department of Transportation, Federal Highway Administration, 1977.

Part 7—Existing Bridges

Page 15-7-6, Article 7.2.1.4. Fasteners. Add the following text:

7.2.1.4 Fasteners

a. Existing rivets that are removed to effect a repair or strengthening shall be replaced on a one for one basis with high strength bolts of equal or greater diameter.

b. Where remaining safe fatigue life is a controlling limit state, existing rivet holes shall be reamed after removal of the rivets, and the replacement high strength bolt shall be one size larger in nominal diameter than the replaced rivet or if of the same diameter shall satisfy the requirements for an oversize hole unless the hole is examined and found to contain no significant flaws or stress raisers.

c. Rivet heads may be removed by either mechanical means or careful use of oxygen-fuel gas cutting methods. If the oxygen-fuel gas method is used, use of a rivet cutting tip is recommended. Where existing material is to be preserved for reuse, rivet shanks shall be removed by mechanical means only, with coring permitted to assist the mechanical removal; the coring process shall not penetrate the surface of the rivet shank. Where existing material is to be discarded, rivets may be removed by any appropriate means acceptable to the Engineer.

d. If a rivet hole has been scored or otherwise damaged, the hole shall be reamed and the replacement high strength bolt shall be one size larger in nominal diameter than the replaced rivet, or if of the same diameter shall satisfy the requirements for an oversize hole.

e. Existing high strength bolts removed to effect a repair or strengthening may be reused only under conditions approved by the Engineer. If unacceptable to the Engineer, they shall be replaced with new high strength bolts of equivalent diameter.

f. The extent of contamination of the faying surfaces by damage, mill scale, paint, grease, etc. shall be considered by the Engineer in assigning the allowable shear values for high strength bolts used in repair, strengthening or retrofitting applications.

g. Type 3 high strength bolts shall be used with weathering steel. Galvanized bolts shall not be used with uncoated steel.

Page 15-7-6, Article 7.2.1.6. Jacking and Temporary Support. Add the following text:

7.2.1.6 Jacking and Temporary Support

a. Jacks shall be placed so that the line of action is as nearly as possible, concentric with the gravity axis of the existing member(s). If jacks must be placed on an eccentric axis, an analysis of the effects of such eccentricity shall be made.

b. The rated capacity of a jack shall be a minimum of 50% greater than the computed required jacking force.

c. When choosing member sizes for jacking, strongbacks, or other temporary support, the allowable stress may be increased by 50%. Attention shall be paid to slenderness ratios and buckling allowables.

Page 15-7-7. Add the following new Article:

7.2.1.8 Heat Straightening

a. Heat straightening may consist of either flame straightening used alone, flame straightening with an auxiliary force, or hot mechanical straightening.

b. Heat straightening of damaged steel members shall not be undertaken by unskilled or inexperienced persons.

c. Heat straightening of damaged steel members shall be undertaken only after due consideration of the stability of the individual member, the stability of the overall structure, and possible redistribution of stresses as a consequence of the heat straightening process.

d. The temperature of the heated steel shall not exceed 1200 F for carbon and low alloy steels, nor 1050 F for quenched and tempered steels.

e. Mechanisms to apply auxiliary forces during heat straightening shall be of the type that reduce the magnitude of these auxiliary forces as the member displaces.

Page 15-7-7. Add the following new Article:

7.2.1.9 Bearings

a. Where expansion bearings are frozen in position by accumulated corrosion, they shall not be freed without prior investigation of the stability of the superstructure and substructure elements.

b. Where a bearing has been pounded into the bearing seat, the bearing may be restored to correct elevation by filling the void under the bearing with a suitable grout. If the restoration of the bearing shoe to correct elevation requires an extension above the seat, steel shim plates may be used.

Part 9—Commentary

Page 15-9-27. Revise Article 9.1.14 as follows, because of the changes to Section 1.14:

9.1.14 Fracture Critical Members

9.1.14.1 Scope (1997)

The implementation of the AWS D1.5-95 Fracture Control Plan for Fracture Critical Members will help to ensure that a steel bridge with critical tension components will serve a useful and serviceable life over the period intended in the original design. Some bridges do not have fracture critical members. However, it is most important to recognize them when they do exist. The Fracture Control Plan should not be used indiscriminately by designers to circumvent good engineering practice.

Section 1.14 covering Fracture Critical Members should be used as an extension of and supplement to the current requirements for welding as specified throughout Chapter 15, Steel Structures and the AWS Specifications. Where not specifically replaced by Section 1.14, all provisions of Chapter 15, Steel Structures and AWS D1.5 still apply.

In 1995, AWS D1.5 was issued, including Section 12 that specifically addresses additional requirements for FCM's. The D1.5-95 code contains the latest provisions to ensure reliable control of weld quality. Major changes from the 1978 AASHTO plan (and modifications) include:

(1) Alternative to lot testing criteria for filler metals (D1.5-95, 12.6.1.1)

(2) Extension of WPS qualification testing period of validity from one to three years (D1.5-95, 12.7.4)

(3) Prequalification of SMAW WPS's for use with electrodes with a tensile strengths 80 ksi (D1.5-95, 12.7.1)

(4) Testing of welding consumables for FCAW and SAW in accordance with AWS A4.3 versus the glycerine method (D1.5-95, 12.6.2.1)

(5) More extensive controls on exposure of welding consumables to atmospheric moisture (D1.5-95, 12.6.5-12.6.7.6)

(6) Preheat levels based upon heat input, diffusible hydrogen levels, as well as steel grade and thickness (D1.5-95, 12.14)

(7) New tack welding requirements (D1.5-95, 12.13)

(8) Qualification testing of WPS's for FCM's is fully consistent with D1.5 requirements for WPS's used on non-fracture critical members (D1.5-95, 12.7)

The following commentary applies to the provisions of D1.5-95 Section 12 FCP as applied to railroad bridges:

Fabricator Qualification Certification [AWS D1.5, Art. 12.8] (1997)

Quality workmanship requires fabrication capability, trained workmen and effective and knowledgeable supervision. The AISC Quality Certification Program evaluates a plant on general management, engineering, drafting, procurement, operations and quality. Each of these areas is divided into sub areas and evaluated for policy statement, organization and personnel, procedures, facilities and equipment, and past record.

Welding Inspector Qualification and Certification [AWS D1.5, Art. 12.16.1.1] (1997)

Although requirements for welder qualification have long been established, little, if anything, was done to determine the competence of welding inspectors. The AWS Standard for Qualification and Certification of Welding Inspectors, QC-1, was developed to ensure that inspection personnel will have the ability to determine if welding is in compliance with requirements of the contract specifications.

Non-Destructive Testing Personnel Qualification and Certification [AWS D1.5, Art. 12.16.1.2] (1997)

Personnel performing non-destructive testing shall be qualified as NDT Level II or Level III, in accordance with ASNT Recommended Practice SNT-TC-1A.

This practice has been upgraded for the non-destructive testing of FCM's by only permitting the testing to be performed by individuals qualified as NDT Level II and working under the supervision of an NDT Level III person or an NDT Level III person to perform the testing. To ensure the capability of the Level III persons, they must be certified by the ASNT or equivalent as determined by the Engineer. The term "under the supervision" is intended to mean that the NDT Level III person will be available, as necessary, and will personally check the NDT Level II person's work on a periodic basis.

Preheat and Interpass Temperatures [AWS D1.5, Art. 12.14] (1997)

The minimum preheat and minimum interpass temperatures require in D1.5-95 are based upon the requirements of the 1978 AASHTO FCP, but modified to incorporate the effects of the heat input of welding, and different levels of diffusible hydrogen in deposited weld metal. The actual minimum preheat and interpass temperature listed on the WPS is selected from the applicable table in D1.5 based upon the grade of steel being welded, the thickness(es) of steel involved, the computed value of welding heat input, and the maximum diffusible hydrogen content in deposited weld metal. While more complex than other systems to determine preheat, this method is considered more accurate and appropriate for fabrication of FCM's.

Welding Consumables [AWS D1.5, Art. 12.6] (1997)

All welding consumables used for fabrication of FCM's must be of controlled quality. D1.5-95 accomplishes this by either requiring lot testing of consumables, or requiring the manufacturer to have a quality assurance program audited and approved by one of the independent agencies listed.

It is not essential that each heat and lot of welding consumables be pretested in the combination that will be actually used in the work. Accepted heats and lots of welding consumables that conform to the same specification are made by the same manufacturer may be interchanged without concern that the weld metal produced will be unacceptable.

Backing [AWS D1.5, Art. 9.2.4.2] (1997)

Steel backing for groove welds using rolled bar stock of limited cross sectional area is considered superior to backing produced by stripping from plate. Bar stock is uniform in cross section and has light mill scale in most instances. Studies of the effects of backing chemistry on weld metal properties indicates that A36 steel is suitable backing for all groove welds in steels with a minimum specified yield stress of 50 ksi or less. The Charpy V-Notch toughness of backing bars of limited dimension will not have a significant influence on the fracture resistance of the groove welds.

It is absolutely essential that all weld backing be continuous and that welds used to join segments of backing be made before the backing is applied to the weld. All joints in backings should be subject to the same weld quality standards and nondestructive tests specified for similar groove welds in the structure.

Welding Procedures [AWS D1.5-95, Art. 12.7] (1997)

Current AWS filler metal specifications recognize the weld metal properties may vary widely, depending on electrode size, flux used, amperage, voltage, plate thickness, joint geometry, preheat and interpass temperature, surface condition, base metal composition and admixture with the deposited metal. Because of the profound effect of the variables, a test procedure is included in these filler metal specifications intending to reproduce "good practice" welding conditions reasonably well and, at the same time, minimize the effect of the more important variables on weld metal properties.

Although the above requirements are adequate for most applications, they are not considered sufficient for Fracture Critical Members. Therefore, D1.5-95 requires all welding procedures to be qualified by test, except for SMAW performed with specific electrodes. This is to help ensure that the weld metal deposited, using the procedure and base metal to be used in production, provides the required toughness.

The deposited weld metal toughness of 25 foot-lb @ –20 degrees F is greater than the toughness of the base metal which it connects. This recognizes the possibilities of discontinuities (porosity, slag inclusions, etc.,) as permitted by AWS D1.5 and that the weld metal may have strength significantly greater than the base metal.

It is the intent of these specifications that a fabricator that has properly completed welding procedure qualification tests within the last 3 years, not be required to repeat the tests for individual railroads unless the railroad has made it a Contract requirement prior to bidding.

The A588 steel test plates and backing specified as an alternative to other steels have been subjected to a metallurgical evaluation that revealed the strength, ductility, and toughness of weld metal produced using this test base metal can be relied upon to indicate whether or not a Welding Procedure Specification will successfully join any of the approved steels with a yield stress of 50 ksi or less. Approval of a single grade of steel will reduce unnecessary testing of base metals and combinations of metals that have no significant effect on the acceptability of the Welding Procedure Specification.

Hydrogen Control [AWS D1.5, Art. 12.6] (1997)

D1.5-95 requires measurement of the SMAW coating moisture, and the diffusible hydrogen content of weld metal deposited by SAW and FCAW. These measurements are made by the filler metal manufacturer. Testing of diffusible hydrogen must be done in accordance with AWS A4.3 which recognizes the latest methods to measure diffusible hydrogen. The glycerine method, previously required by the AREA FCP, is no longer permitted as the results obtained by this method were highly variable, often resulting in artificially low values. The A4.3 methods are more consistent and more accurately represent actual hydrogen values.

Welder Qualification [AWS D1.5, Art. 12.8.2] (1997)

It is intended that welders, welding operators, and tackers be qualified by test within six months prior to the start of fabrication or regularly requalified on the annual basis. Once welders are qualified on the basis of mechanical and radiographic tests, yearly examination of radiographs is considered an acceptable method of assuring that welders and welding operators remain qualified.

Repair Welding [AWS D1.5, Art. 12.17] (1997)

Repair welding consists of deposition of additional weld metal to correct a surface condition, such as insufficient throat or undercut, or procedures which require removal of weld or base metal preparatory to correcting defects in materials or workmanship. The latter are divided into non critical and critical repairs as determined by type and size of defect.

Because virtually all weld repairs are made under conditions of high restraint, the minimum preheat/interpass temperatures requirements are generally higher than specified for the original welding. In addition, the minimum preheat for the repair area must be continued after completion of the repair until a post weld heating of 400 F to 500 F has been completed. This post weld heating is to enhance diffusion of any hydrogen that might be present.

Again, because of the possible high restraint situation in repair welding, a longer time interval is required between completion of the weld repair and final non-destructive testing than in original welding.

Documentation of both non-critical and critical repair welding is required. This is to enable these areas to be given special attention when inspections are made after the bridge is in service.

Non-Destructive Testing of Fracture Critical Members [AWS D1.5, Art. 12.16] (1997)

The FCP recognizes that control of quality is the responsibility of the fabricator. However, it is the prerogative of the Engineer to assure himself that the quality of the product is as specified. This latter includes Quality Assurance (QA), witnessing of Quality Control (QC), testing, review of the fabricator's documentation of his visual and non-destructive testing and duplicating any such work as is deemed necessary. For production schedules to be maintained, it is essential that all QA work be carried out in a timely manner to minimize interference with production.

The effectiveness of radiographic testing and ultrasonic testing is determined by the size, shape and orientation of discontinuity. The Plan requires that both methods of testing be used in determining the quality of all transverse tension groove welds. When the configuration of the material utilizing tension groove welds is not in the same plane, only ultrasonic testing is required.

The penetrating power and intensity of X-rays can be controlled by the user, but these same factors cannot be controlled by the user of gamma rays. The penetrating power of cobalt 60, 1.2 and 1.3 MEV, is so much higher than required for material thicknesses normally used in bridge construction that it is difficult to discern the small changes in thickness due to discontinuities. Iridium 192, however, has a lower and broader equivalent voltage, 0.2 to 0.6 MEV, and more closely approaches the operating characteristics of X-rays. Therefore, the use of cobalt is restricted to material thickness over 3 inches and is permitted because available sources for the thicker material are limited.

9.1.14.2 Definitions (1997)

a. Fracture Critical Members (FCM) are defined as those tension members or tension components of members whose failure would be expected to result in collapse of the bridge or inability of the bridge to perform its design function. The identification of such components must of necessity be the responsibility of the bridge designer since virtually all bridges are inherently complex and the categorization of every bridge and every bridge member is impossible. However, to fall within the fracture critical category, the component must be in tension. Further, a fracture critical member may be either a complete bridge member or it may be a part of a bridge member.

b. Some examples of critical complete bridge members are girders of two-girder bridges and tension chords in truss bridges, provided a failure would cause loss of serviceability of the bridge. Some bridges do not depend on any single member, be it in tension or in compression, for structural integrity. Critical tension components of structures usually occur in flexural members. The tension flange of a flexural member is a critical component if a failure of the specific flexural member would cause loss of serviceability of the bridge. The web of a flexural member, adjacent to the tension flange, can be a critical component.

c. Members or member components whose failure would not cause the bridge to be unserviceable are not considered fracture critical. Compression members and member components in compression may in themselves be critical but do not come under the provisions of this Plan. Compression components do not fail by crack formation and extension by rather by yielding or buckling. Similarly, riveted and bolted members, even though in tension, may not come under the provisions of this Plan. The Plan provides for additional quality of material and provides for increased care in the fabrication and use of the materials to lessen the probability of fracture of tension components from crack formation and extension under static and fatigue loading.

9.1.14.3 Design and Review Responsibilities (1997)

a. A critical part of any complete Fracture Control Plan must deal with design and detailing. These two sections are not addressed in Section 1.14, Fracture Critical Members or in this commentary primarily because they are already included in other parts of this chapter. Fatigue requirements are extensively covered in Article 1.3.13 and, where necessary, are made more conservative for fracture critical members (see Table 1-21). Fatigue categories for various bridge details also are extensively covered. However, it remains a prime responsibility of the designer to examine each detail in the bridge for compliance with the fatigue requirements and to ensure that the detailing will allow effective joining techniques and non-destructive-testing of all welded joints. It is emphasized that the Fracture Control Plan must begin with the designer and that without proper design, details and specifications, the Plan will fail.

b. The designer is the only one with sufficient knowledge of the design to determine if fracture critical members are present and to specifically delineate those members or member components. It is, therefore, his responsibility to designate on the plans those members or member components which are fracture critical. Further, he also is responsible for the review of the shop drawings to determine whether the plans and specifications have been properly interpreted and that the fracture critical members are identified and properly fabricated.

9.1.14.5 Notch Toughness of Steel in Fracture Critical Members (1997)

For comments relating to Table 1-15, see Article 9.1.2.1.

The notch toughness requirements for steels in railroad bridges are similar to those used in steel highway bridges as specified by AASHTO. The requirements developed by AASHTO were adopted after considerable research and deliberation between representatives of the AASHTO Subcommittees on Bridges and on Materials, the Federal Highway Administration, the American Iron and Steel Institute, the American Institute of Steel Construction and various consultants. These requirements were based on numerous technical considerations that include the following:

(1) An understanding of the effects of constraint and temperature on the fracture toughness behavior of steels that were established by testing fracture mechanics specimens.

(2) An understanding of the effects of rate of structural loading on the fracture toughness behavior of structural steels.

(3) The development of a correlation between impact fracture toughness values (K_{Id} obtained by testing fracture toughness type specimens under impact loading) and impact energy absorption for Charpy V-notch (CVN) impact specimens.

(4) Specification of CVN impact toughness values that ensure elastic-plastic initiation behavior for fracture of fatigue cracked specimens subjected to minimum operating temperatures and maximum in-service rates of loading.

(5) A verification of the selected toughness values by the testing of fabricated bridge girders that were subjected to the maximum design fatigue life, followed by testing at the minimum operating temperature and the maximum in-service rate of loading.

(6) An awareness of the extensive satisfactory service experience with steels in bridges and an understanding of the factors that have occasionally led to brittle fractures in bridges.

The safety and reliability of steel bridges are governed by material properties, design, fabrication, inspection, erection and usage. Both AASHTO and AREA recognize that attention to all of these factors is essential and that excessive attention to any single item will not necessarily overcome the effects of a deficiency in any other item.

Neither the AASHTO nor the AREA fracture toughness requirements are sufficient to prevent brittle fracture propagation under certain possible combinations of poor design, fabrication or loading conditions. To accomplish that fact would require intermediate or upper-shelf dynamic toughness levels (also called crack arrest) and these levels of fracture-toughness are not needed to ensure the safety and reliability of the steel bridges.

The general difference in initiation and propagation behavior as related to fracture toughness test results is shown schematically in Figure 9-3. The curve labeled "static" refers to the fracture toughness obtained in a K_{Id} test under conditions of slow loading. (The curve for intermediate loading rate tests, which are extremely complex to run, would be shifted slightly to the right of the static curve. The AASHTO material toughness requirements were developed using an intermediate loading rate

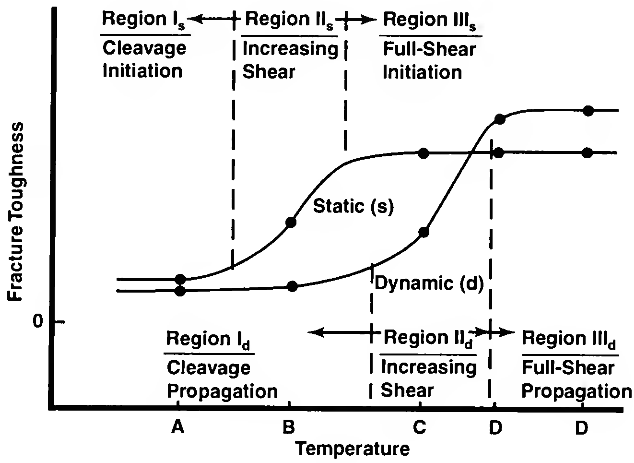


Figure 9-3
Schematic Showing Relation Between Static and Dynamic Fracture Toughness

found applicable to actual bridge structures.) The impact curve is from K_{Id} or other dynamic test under conditions of impact loading. The difference between these two is the temperature shift, which is a function of yield strength for structural steels. In an actual structure loaded at temperature A, initiation may be static and propagation dynamic. However, there is no apparent difference between the two because both initiation and propagation is by cleavage. If a similar structure is loaded slowly to failure at temperature B, there will be some localized shear and a reasonable level of static fracture toughness at the initiation of failure. However, for rate sensitive materials, such as structural steels used in bridges, once the crack has initiated, the notch toughness is characterized by the dynamic toughness level on the impact curve and the fracture appearance for the majority of the fracture service is cleavage. If the structure is loaded slowly to fracture initiation at temperature C, the initiation characteristics will be full shear initiation with a high level of plane stress, crack toughness K_c. However, the fracture surface of the running crack may still be predominately cleavage but with some amount of shear as shown in the lower impact curve at temperature C in Figure 9-3.

The use of impact or dynamic fracture tests in fracture control would predict no difference in actual resistance to fracture between temperatures A and B and only a modest difference between B and C. In fact, however, there is a considerable increase in resistance to fracture initiation between A and B and between B and C, as is indicated by slow loading tests such as K_{Ic} or crack opening displacement tests. However, there is essentially no difference in the resistance to fracture propagation (i.e. crack arrest behavior) between A and B, and the difference between B and C is modest. Thus, to prevent brittle fracture propagation in a structure by using material toughness alone (i.e. without proper control of design, fabrication, inspection and usage), the impact toughness must be quite high, e.g. approaching full shear propagation behavior temperature D. Even then, there may be situations where crack growth still occurs.

In summary, application of the AREA material toughness requirements should provide a high level of elastic-plastic or plastic initiation behavior for steels with fatigue cracks loaded to maximum in-service rates of loading at the minimum service temperature. Because the AREA Fracture Control Plan addresses all aspects that may lead to brittle fractures or fatigue failure (i.e. material properties, design, fabrication, inspection, erection and usage), these material toughness requirements should be satisfactory in the context of the total AREA Fracture Control Plan.

9.1.14.6 Fatigue (1997)

For Fracture Critical Members, the five steps outlined in Article 9.1.3.13 and summarized in Table 9-1 were repeated, using a modified value of α. Since α was introduced as a beneficial reduction factor reflecting the participation of bracing, floor system and the three-dimensional response of the structure and the fact that full impact does not occur for each stress cycle, it was increased by a constant multiplier factor of 1.2 for Fracture Critical Members. This reduces the beneficial effect of secondary components and creates a more severe stress cycle for these conditions. Rather than introducing a change in both stress cycles and its corresponding stress range, only the decreased stress range values corresponding to the increased constant cycles that result from the factor 1.2 α are shown in Table 1-21. The design stress cycles shown in Table 1-7 are to be used with the stress range values provided in Table 1-21.

Page 15-9-41. Revise the Welding Index as follows, to reflect changes to Section 1.14 and Article 9.1.14.

Welding Index (1997)

This Welding Index makes reference to some of the articles in the Manual pertaining to Welding involved in design, fabrication, repair and rating of steel structures. This index does not include every reference to welding within the Manual, but can serve as a ready guide for designers.

Subject	Article Reference
Bridge welding code, AWS D1.5	1.2.2; 1.10.2; 1.10.6; 1.14; 3.3.1a; 3.3.5; 3.3.5b; 9.1.2.2; 9.1.4.2; 9.3.1.6
Welding: requirements	1.2.2; 3.3.1; 1.14.4
(Balance of Index unchanged)	

Proposed 1997 Manual Revisions to Chapter 19—Bridge Bearings

Page 19-i. Replace the current outline for Chapter 19 with the following full text version:

AMERICAN RAILWAY ENGINEERING ASSOCIATION

MANUAL FOR RAILWAY ENGINEERING

CHAPTER 19

BRIDGE BEARINGS*

(Responsibility for this Chapter lies with Committee 15 - Steel Structures)

Current From August 1, 1997 to July 31, 1998

Specification for Railway Bridge Bearings

FOREWORD

Parts 1 and 2 formulate specific and detailed rules for the design and construction of bearings for nonmovable railway bridges. Part 3 is a commentary, including bibliography, for explanation of various articles in the preceding parts.

TABLE OF CONTENTS[1]

Part 1—Design (1997)

*The material in this and other chapters in the AREA Manual for Railway Engineering is published as recommended practice to railroads and others concerned with the engineering, design and construction of railroad fixed properties (except signals and communications), and allied services and facilities. For the purpose of this Manual, RECOMMENDED PRACTICE is defined as a material, device, design, plan, specification, principle or practice recommended to the railways for use as required, either exactly as presented or with such modifications as may be necessary or desirable to meet the needs of individual railways, but in either event, with a view to promoting efficiency and economy in the location, construction, operation or maintenance of railways. It is not intended to imply that other practices may not be equally acceptable.

[1]Latest page consist: i to iii incl. (1997)

Part 1

Design

1997

FOREWORD

The purpose of this part is to formulate specific and detailed rules for the design of bearings for nonmovable railway bridges.

CONTENTS

1.1 INTRODUCTION

1.1.1 Definition of Terms

1.1.1.1 Contractual Terms

a. The term "Company" means the railway company party to the contract. The term "Engineer" means the chief engineering officer of the Company or his authorized representatives. The term "Inspector" means the inspector representing the Company. The term "Contractor" means the manufacturing, fabricating or erecting contractor party to the contract.

b. See Section 1.1 of AREA Manual Chapter 15 for other contractual terms and/or requirements for "Proposals," "Shop Drawings," "Drawings to Govern," "Patented Devices" and "Notice to Engineer."

1.1.1.2 Bearing Component Terms

a. The following technical terms are used throughout this Chapter for the various types or components of Bridge Bearings:

Base Plate: A steel plate, whether cast, rolled or forged, usually used to uniformly distribute line bearing loads from a rocker, rocker plate, roller, or roller nest to other bridge bearing components.

Bed Plate: See Masonry Plate or Base Plate.

Bronze or Copper-Alloy Sliding Expansion Bearing: (UNDER DEVELOPMENT)

Bolster: A block-like member composed of wood, metal, or concrete used to transmit and distribute a bridge bearing load to the top of a pier cap or abutment bridge seat; or to raise a bridge bearing above moisture or debris that may collect on a bridge seat. Metal bolsters frequently consist of voided iron or steel castings, or built up steel weldments.

Disc Bearing: A type of multi-rotational bearing which provides for end rotation of bridge spans by means of a flat circular shaped elastomeric disc.

Elastomeric Bearing: A device constructed partially or wholly from elastomer for the purpose of transmitting loads and accommodating movement between a bridge span and its supporting structure.

External Steel Load Plate: A steel plate bonded to the upper and/or lower surfaces of an elastomeric bearing.

Guide Bar: (UNDER DEVELOPMENT)

Guide Key: (UNDER DEVELOPMENT)

Masonry Plate: A steel plate or plate-shaped member, whether cast, rolled or forged, usually placed upon a masonry pier, abutment or other substructure unit and used to distribute loads from upper components of a bridge bearing uniformly over the masonry bridge seat below.

Multi-Rotational Bearing: A type of bearing or bearing device which has the capability of allowing rotation in any of several directions, typically both transverse and longitudinal directions. Multi-rotational bearings frequently include a circular elastomeric disc or pad, or spherical sliding surface.

Pedestal: A block-like member or assemblage composed of wood, metal, or concrete used to transmit and distribute a load from a bridge bearing or other member or part of a structure to another member or part. Metal pedestals frequently consist of voided iron or steel castings, or built up steel weldments.

Pin: A cylindrical bar, usually steel, used to connect, hold in position, and/or transmit loads from one bridge bearing component to another while allowing for the rotation of those bridge bearing components relative to each other.

Pintle: (UNDER DEVELOPMENT)

Plain Elastomeric Bearing: An elastomeric bearing that consists of elastomer only.

Pot Bearing: A type of bearing which usually consists of an elastomeric disc confined in a steel cylinder, or pot, with a ring sealed steel piston which transmits bridge bearing loads to the elastomeric disc.

Reinforced Elastomeric Bearing: An elastomeric bearing that consist of layers of elastomer restrained at their interfaces by integrally bonded steel reinforcement.

Rocker: A cylindrical sector shaped member attached, frequently by a pin at its axis location, to the expansion end of a girder or truss that will transmit bridge bearing loads in line bearing contact upon its perimetrical surface with a base plate, bolster, pedestal or masonry plate and thus provide for longitudinal movements by a wheel-like translation.

Rocker Plate: A steel plate with one cylindrical surface that will transmit bridge bearing loads in line bearing contact upon its perimetrical surface to other bearing components and allow for longitudinal rotation of the span ends due to span deflection.

Roller: A steel cylindrical shaped member, frequently forming an element of a roller nest or any other bearing device intended to provide longitudinal movements by rolling contact and that will transmit bridge bearing loads in line bearing contact with both a top plate or sole plate above, and a base plate, bolster, pedestal or masonry plate below.

Roller Nest: A group of two or more steel cylinders forming a part of an expansion bearing at the movable end of a girder or truss intended to provide longitudinal movements by rolling contact and that will transmit bridge bearing loads in line bearing contact with both a top plate or sole plate above, and a base plate, bolster, pedestal or masonry plate below. Commonly, the rollers of a roller nest are assembled in a frame or box.

Seismic Isolation Bearing: A type of bridge bearing which is intended to reduce the dynamic response of a bridge superstructure and thus minimize seismic loads acting on, and damage to, the bridge by providing a compliant connection between the superstructure and substructure through viscous damping, friction or metallic yielding.

Seismic Isolation Device: A device which is intended to reduce the dynamic response of a bridge superstructure and thus minimize seismic loads acting on, and damage to, the bridge by providing a compliant connection between the superstructure and substructure through viscous damping, friction or metallic yielding. A seismic isolation device may be a component of a seismic isolation bearing or may be a device, or one of several devices, independently connected between the bridge superstructure or substructure.

Shoe: A bolster-like or pedestal-like member or plate, typically placed under the end of a plate girder or truss, to transmit and distribute bridge bearing loads to the masonry bridge seat, other bearing components or to other substructure members.

Sole Plate: A steel plate bolted, riveted, or welded directly under the bottom flange of a rolled beam or plate girder, bottom chord of a truss, or cast into the bottom of a concrete girder, to uniformly distribute the bridge bearing loads into other bridge bearing components below, such as a roller nest, rocker plate, base plate, pedestal, multi-rotational bearing or masonry plate.

Spherical Bearing: A type of multi-rotational bearing which provides for end rotation of bridge spans by means of a convex spherical surface hinging, rocking or sliding in a mating concave spherical surface. Lubrication of the mating surfaces is usually required and is frequently accomplished by providing a TFE Bearing Surface or a Bronze or Copper-Alloy Sliding Surface.

TFE Bearing Surface: A low-friction sliding surface which utilizes a polytetrafluorethylene (TFE) sheet or woven fiber fabric manufactured from pure virgin unfilled TFE resin which is bonded to a steel backing substrate and usually slides against a polished stainless steel sheet.

1.1.2 General Requirements

a. Design of bearings shall be such as to allow for expansion and contraction of the spans resulting from change in temperature at the rate of 25 (1 in.) in 30,000 (100 ft.) for Minimum Service Temperature[1] Zone 1 and 30 (1¼ in.) in 30,000 (100 ft.) for Minimum Service Temperature Zones 2 and 3. Provisions shall also be made for change in length of the span resulting from live load. In steel spans more than 90,000 (300 ft.) long, allowance shall be made for expansion of the floor system. Due consideration shall be given to the effects of lateral thermal movement for structures wider than 12,000 (40 ft.).

b. Bearings and ends of spans shall be securely anchored against lateral and vertical movement. The Engineer may waive the requirement for vertical restraint of concrete spans.

[1]Minimum Service Temperature Zones 1, 2 and 3 are defined in AREA Manual Chapter 15, Article 9.1.2.1 and illustrated in Figure 9.1.2.1A.

c. Bearings for spans of less than 15,000 (50 ft.) need not have provision to accommodate rotation due to deflection of the span.

d. Bearings for spans of 15,000 (50 ft.) or greater shall have provision to accommodate rotation due to deflection of the span. This requirement can be accommodated by use of a type of bearing employing a hinge, curved bearing plate or rocker plate, elastomeric pad, or pin arrangement.

e. End bearings subject to both longitudinal and transverse rotation shall consist of elastomeric or multi-rotational bearings.

f. Due consideration shall be given to bearing stability under seismic loading in the selection of bearing type.

g. Bearings on masonry preferably shall be raised above the bridge seat by masonry plates, pedestals or bolsters. The Engineer may waive this requirement for elastomeric bearings.

h. Provision for the replacement of bearings shall be considered in the design.

1.1.3 Expansion Bearings

a. The expansion end of spans of 21,000 (70 ft.) or less may be designed to accommodate movement through the use of low friction sliding surfaces or elastomeric pads.

b. The expansion end of spans longer than 21,000 (70 ft.) shall be supported by bearings employing rollers, rockers, reinforced elastomeric pads, or low friction sliding surfaces designed to accommodate larger longitudinal movements.

1.1.4 Fixed Bearings

a. The fixed end of spans shall be securely anchored to the substructure to prevent lateral and longitudinal movement.

b. Span rotation shall be accommodated as stipulated in the provisions of Articles 1.1.2 c, 1.1.2 d and 1.1.2 e.

1.1.5 Bearing Selection Criteria

(UNDER DEVELOPMENT)

1.2 BASIC ALLOWABLE STRESSES

a. The basic allowable stresses to be used in proportioning the parts of a bridge bearing shall be as specified below.

1.2.1 Structural Steel, Bolts and Pins

a. Except as provided in c below, the basic allowable stresses for all steel bearing components, weld metal, bolts or rivets, shall be as specified in Section 1.4 of AREA Manual Chapter 15.

b. When the allowable stress for steel bearing components is expressed in terms of F_y, F_y = yield point of the material as specified in AREA Manual Chapter 15, Table 1.2.1A.

	MPa (psi)
c. Bearing on pins	$0.75F_y$
F_y = yield point of the material on which the pin bears, or of the pin material, whichever is less, as specified in AREA Manual Chapter 15, Table 1.2.1A.	
Bearing on milled web members, milled stiffeners and other steel parts in contact, except as specified in this Article	$0.83F_y$
Bearing between rockers and rocker pins	$0.375F_y$

F_y = yield point of the material in the rocker or in the rocker pin, whichever is less, as specified in AREA Manual Chapter 15, Table 1.2.1A.

Stress in extreme fibers of pins		$0.83F_y$
	kN/mm	(Pounds per linear inch)
Bearing on rollers, rockers and rocker plates:		
For diameters up to 600 (25 in.)	$\frac{(F_y - 90)d}{33{,}000}$	$\left(\frac{F_y - 13{,}000)600d}{20{,}000}\right)$
For diameters from 600 (25 in.) to 3000 (125 in.)	$\frac{(F_y - 90)\sqrt{d}}{1300}$	$\left(\frac{F_y - 13{,}000)3000\sqrt{d}}{20{,}000}\right)$

d = diameter of roller, rocker, or rocker plate curved surface; mm (in.)

F_y = yield point of the material in the roller, rocker, or rocker plate; or in the base plate, whichever is less, as specified in AREA Manual Chapter 15, Table 1.2.1A.

1.2.2 Cast Steel

a. For cast steel, the allowable stresses in compression and bearing shall be the same as those allowed for structural steel with the same yield point or yield strength. Other allowable stresses shall be 75 percent of those allowed for structural steel with the same yield point or yield strength.

1.2.3 Bronze or Copper-Alloy Plates

a. For self-lubricating bronze or copper-alloy plates, the allowable stresses in bearing on the net area shall not exceed 14 MPa (2,000 psi).

1.2.4 Elastomeric Bearings

a. For unconfined elastomeric bearings, the allowable average compressive stress shall be as specified in Article 1.6.3.4, but shall not exceed 7 MPa (1.000 psi) for reinforced bearings, or 5.5 MPa (800 psi) for plain bearings.

1.2.5 Masonry

a. Except as provided in (b) below, the basic allowable stresses for all concrete masonry and reinforcing steel shall be as specified in AREA Manual Chapter 8.

b. Bearing pressure:	MPa (psi)
Granite	5.5 (800)
Sandstone and limestone	3 (400)
Concrete	0.25 of the ultimate compressive strength

(When the strength of concrete is unknown, use 17 MPa (2,500 psi) for the design ultimate compressive strength.)

1.2.6 Timber

a. Allowable bearing stresses and other design stresses for all timber members shall be as specified in AREA Manual Chapter 7.

1.3 STEEL BEARING COMPONENTS

1.3.1 Scope

a. This Section covers the materials for, and the design of, steel bearings or steel bearing components made from rolled steel plates and shapes, steel forgings or cast steel which are used to carry railroad loading. The use of ductile or malleable iron castings is also covered for specific bearing components.

b. The fabrication and installation of steel bridge bearings or steel bearing components shall be in accordance with the requirements of Section 2.2 of this Chapter and AREA Manual Chapter 15.

1.3.2 Materials

a. Except as provided in Articles 1.3.2.1 through 1.3.2.4 below, material for all bearing components produced from rolled steel plates or shapes, shall conform to one of the ASTM designations listed in AREA Manual Chapter 15, Table 1.2.1A.

1.3.2.1 Pins, Rollers, and Rockers

a. In addition to the designations listed in AREA Manual Chapter 15, Table 1.2.1A, steel for pins, rollers, and rockers may conform to one of the ASTM A 668M (A 668) classes: Steel Forgings, Carbon and Alloy, for General Industrial Use, listed below in Table 1.3.2.

1.3.2.2 Fasteners—Rivets and Bolts

a. Fasteners may be carbon steel bolts, ASTM A 307, power-driven rivets, ASTM A 502 Grades 1 or 2, or high-strength bolts, ASTM A 325M (A 325) or ASTM A 490M (A 490) in accordance with AREA Manual Chapter 15.

1.3.2.3 Weld Metal

a. Weld metal shall conform to the current requirements of AWS D1.5 and AREA Manual Chapter 15.

1.3.2.4 Cast Steel, Ductile Iron Castings and Malleable Castings

1.3.2.4.1 Cast Steel and Ductile Iron

a. Cast steel shall conform to specifications for Steel Castings for Bridges, ASTM A 486; Mild-to-Medium-Strength Carbon-Steel Castings for General Application, ASTM A 27M (A 27); and Corrosion-Resistant Iron-Chromium, Iron-Chromium-Nickel and Nickel-Based Alloy Castings for General Application, ASTM A 743M (A 743).

b. Ductile iron castings shall conform to ASTM A 536.

Table 1.3.2. Minimum Material Properties—Pins, Rollers, and Rockers

ASTM A 668M (A 668) Class	F_y – Min Yield Point, MPa (psi)	Size Limitation[1]
Class C	230 (33,000)	To 500 (20 in.) in dia.
Class D	260 (37,500)	To 500 (20 in.) in dia.
Class F	345 (50,000)	To 250 (10 in.) in dia.
Class G[2]	345 (50,000)	To 250 (10 in.) in dia.

[1]Expansion rollers shall be not less than 150 (6 in.) in diameter.
[2]Rolled material of the same properties may be substituted.

1.3.2.4.2 Malleable Castings

a. Malleable castings shall conform to specifications for Malleable Iron Castings, ASTM A 47M (A 47), Grade 22010 (32510) {minimum yield point, Fy, 220 MPa (32,500 psi)}.

1.3.3 Shoes and Pedestals

a. Shoes and pedestals shall be designed on the assumption that the vertical load is distributed uniformly over the entire bearing surface. They shall be either made of cast steel or built-up by welding rolled steel and/or cast steel elements together.

b. Shoes and pedestals preferably shall be made of cast steel or structural steel. No part of a cast steel shoe, and no load carrying part of a welded shoe, shall be less than 25 (1 in.) thick.

c. In a welded shoe, the vertical load shall be carried directly in bearing between elements. Diaphragms shall be provided between web surfaces to ensure stability of component parts.

d. The difference in width and length between top and bottom bearing surfaces shall not exceed twice the vertical distance between them. For hinged bearings with pins, the vertical distance shall be measured from the centerline of pin.

e. Webs and pin holes in the webs shall be arranged to keep any eccentricity to a minimum. The net section through the hole shall provide 140 percent of the net section required for the actual stress transmitted through the pedestal or shoe. Pins shall be of sufficient length to ensure a full bearing. Pins shall be secured in position by appropriate nuts with washers. All portions of shoes and pedestals shall be secured against lateral movement of the pins.

1.3.4 Rocker Plates, Rockers and Rollers

1.3.4.1 Rocker Plates

a. Rocker plates shall be used in preference to either rockers or rollers where conditions permit. Rocker plates must be used in conjunction with a base plate. For expansion bearings, the base plate must be supported on a reinforced elastomeric pad or low friction sliding surface designed to accommodate longitudinal movements from both temperature and translations at the span bearing elevation resulting from span rotations due to span deflections. The elastomeric pad or sliding surface should only be placed at the expansion end of the span and must be designed to also accommodate the additive longitudinal movements resulting from fixed end span rotation.

b. The nominal centerline thickness of the rocker plate shall not be less than 40 (1-1/2 in.) and the curved surface shall be cylindrical.

c. The rocker plate shall be doweled to the base plate to prevent lateral movement, skewing or creeping of the rocker plate on the base plate.

d. The effective length of rocker plate, for calculating line bearing stress, shall be the lesser of the actual length of the rocker plate or the least length determined as follows:

1. The length of any bearing surface above or below the rocker plate, such as the bottom flange width or length of sole plate for steel spans, plus the total thickness of all plates between the bearing surface and the line bearing or curved surface of the rocker plate (including the nominal centerline thickness of the rocker plate if between the two surfaces).

2. For steel spans, the distance out-to-out of bearing stiffeners plus the total thickness of all plates between the bottom of bearing stiffener surface and the line bearing or curved surface of the rocker plate (including the bottom flange thickness and the nominal centerline thickness of the rocker plate if between the two surfaces).

3. For concrete spans, the length of sole plate cast into the bottom of the span plus the total thickness of all plates between the bottom of sole plate bearing surface and the line bearing or curved surface of the rocker plate (including the nominal centerline thickness of the rocker plate if between the two surfaces).

e. The effective length of rocker plate, for calculating line bearing stress, shall also be reduced by the diameter of each dowel hole located on the line bearing surface.

1.3.4.2 Rockers

a. Rockers shall be used in preference to rollers where conditions permit. The upper surface of rockers shall have a pin or cylindrical bearing.

b. The lower portion of a rocker, at the nominal center line of bearing, shall not be less than 40 (1½ in.) thick and the lower surface shall be cylindrical with its center of rotation at the center of rotation of the upper bearing surface.

c. The rocker shall be doweled to the base plate to prevent lateral movement, skewing or creeping of the base of the rocker.

d. The effective length of rocker, for calculating line bearing stress, shall not be greater than the length of the upper bearing surface plus the distance from the lower surface to the upper bearing surface. There shall be sufficient web material between the upper and lower portion of the rocker to ensure uniform distribution of load over the effective length of rocker.

e. The effective length of rocker, for calculating line bearing stress, shall also be reduced by the diameter of each dowel hole located on the line bearing surface.

1.3.4.3 Rollers

a. Rollers may be either cylindrical or segmental and shall not be less than 150 (6 in.) in diameter.

b. Rollers shall be connected by substantial side bars to ensure parallelism and shall be guided by gearing or other effective means to prevent lateral movement, skewing, and creeping.

c. Rollers and bearing plates shall be protected from dirt and water as far as practicable, and the design shall be such that water will not be retained and the roller nests will be accessible for inspection and cleaning.

1.3.5 Sole, Base and Masonry Plates

a. Base and masonry plates shall be designed on the assumption that the vertical load is distributed uniformly over a bearing area with effective length and width as defined in (b), except for eccentricity from rocker travel.

b. The effective length of the bearing area shall not be greater than the effective length of the rocker plate or rocker as defined in Articles 1.3.4.1(c) or 1.3.4.2(c), or the length of the roller, plus 2 times the thickness of the base plate. The effective width of the bearing area shall not be greater than 4 times the thickness of the base plate for a single roller, rocker plate or rocker, or the distance between multiple rollers plus 4 times the thickness of the base plate for roller nests.

c. For spans designed to slide on bearings with smooth surfaces without hinges, the distance from centerline of bearing to edge of masonry plate, measured parallel with the track, shall not be more than 2 times the thickness of the plate plus 100 (4 in.).

d. Sole plates shall have a minimum thickness of 20 (¾ in.).

e. Base and masonry plates shall have a minimum thickness of 40 (1½ in.).

1.3.6 Inclined Bearings

a. For spans on an inclined grade and without hinged bearings, the sole plates shall be beveled so that the bottom of the sole plate is level, unless the bottom of the sole plate is radially curved. All other bearing plate and masonry surfaces shall be made level.

1.3.7 Anchor Bolts and Rods

a. Provision for anchorage of spans to the substructure shall be as follows:

1. Steel trusses, steel girders, steel rolled beam spans, steel masonry plates and timber beams shall be securely anchored to the substructure with anchor bolts.

2. Except when waived by the Engineer, concrete spans shall be anchored to the substructure with anchor bolts or anchor rods.

b. Except as provided in (e) below, anchor bolts shall not be less than 32 (1¼ in.) diameter. There shall be washers under the nuts. Anchor bolt holes in pedestals, masonry plates, or sole plates shall be 10 (⅜ in.) larger in diameter than the bolts. At expansion bearings the holes in the sole plates shall be slotted.

c. Except as provided in (e) below, anchor bolts and anchor rods that are not required to resist uplift shall extend at least 300 (12 in.) into the masonry. Those that are required to resist uplift shall be designed to engage a substantial mass of masonry, the weight of which is at least 1.5 times the uplift.

d. Anchor bolts and anchor rods may be cast into the substructure concrete or may be installed in holes drilled into the masonry substructure. Anchor bolts and anchor rods shall be swedged, threaded or shall have rolled deformations to secure a satisfactory bond with the material in which they are embedded.

e. The following are minimum requirements for anchorage at each bearing, except as noted:

1. For steel or timber spans:

For steel rolled beam spans:

The outer beams under each track shall be anchored at each end with 2 anchor bolts.

For timber beams:

Spans 15,000 (50 ft.) in length or less: In accordance with AREA Manual Chapter 7.

For steel trusses, steel girders, or timber beams:

Spans 15,001 (51 ft.) to 30,000 (100 ft.): 2 anchor bolts.

Spans 30,001 (101 ft.) to 46,000 (150 ft.): 2 anchor bolts, 40 (1½ in.) in diameter, embedded 400 (15 in.) in the masonry.

Spans greater than 46,000 (150 ft.): 4 anchor bolts, 40 (1½ in.) in diameter, embedded 400 (15 in.) in the masonry.

2. For concrete spans:

For slab and double box beams: 2 anchor rods per slab or beam end.

For multiple single box beams: 1 anchor rod per beam end.

For I-beams: 2 anchor bolts per beam end.

1.3.8 Central Guide Keys and Guide Bars

a. Central guide keys may be made integral by machining from the solid. Where a separate key or guide bar is used it shall be fitted in a keyway slot machined into the plate to give a press fit and bolted or welded to the plate to resist overturning.

b. Guide bars may be made integral by machining from the solid or may be fabricated from bars and welded or bolted to resist overturning.

c. Guide bars and central guide keys shall be designed for the specified horizontal forces, but not for less than 10% of the vertical capacity of the bearing.

d. The total clearance between the key/guide bars and guided members shall be 3 (1/8 in.) maximum.

e. Guided members must have their contact area within the guide bars in all operating positions.

f. Guiding off the fixed base or any extensions of it, where transverse rotation is anticipated, shall be avoided.

1.4 BRONZE OR COPPER-ALLOY SLIDING EXPANSION BEARINGS

(UNDER DEVELOPMENT)

1.5 TFE BEARING SURFACE

(UNDER DEVELOPMENT)

1.6 ELASTOMERIC BEARINGS

1.6.1 Scope

a. This Section covers the materials for, and the design of, plain and reinforced elastomeric bearings made from polyisoprene (natural rubber) or polychloroprene (Neoprene) which are used to carry railroad loading.

b. Cushioning pads, used to provide a smooth even bearing surface, are not covered in this specification since they are very thin and are not designed to translate or to withstand the loads without failure.

c. The fabrication and installation of elastomeric bridge bearings shall be in accordance with the requirements of Section 2.5 of this Chapter.

1.6.2 Materials

1.6.2.1 Elastomer

a. The elastomeric compound shall be specified by the Engineer and shall be 100 percent virgin polyisopene (natural rubber), or virgin crystallization resistant polychloroprene (Neoprene) meeting the requirements of Table 1.6.2. When test specimens are cut from the finished product, a ten percent variation in "Physical Properties" shall be allowed.

b. Material with a nominal hardness greater than 60 durometer shall not be used in reinforced bearings.

1.6.2.2 Steel Reinforcement

a. Steel reinforcement for reinforced elastomeric bearings shall be not less than 1.5 (0.0598 in.) thick, and shall be rolled from mild steel sheet conforming to ASTM A 570M (A 570), Grade 36; or ASTM A 611, Grade D unless otherwise specified by the Engineer.

1.6.2.3 External Steel Load Plates

a. Elastomeric bridge bearings may have external steel load plates bonded to the upper and/or lower surfaces. Such load plates shall be at least as large as the elastomer layer to which they are bonded. Steel load plates shall be tapered, if necessary, to ensure full bearing contact between non-parallel load surfaces. Tapered layers of elastomer are not permitted.

b. External steel load plates shall meet the requirements of AREA Manual Chapter 15, Part 1, except as modified by Section 1.3 of this Chapter.

1.6.3 Design

a. This section covers the design of plain pads (consisting of elastomer only) and reinforced bearings (consisting of layers of elastomer restrained at their interfaces by integrally bonded steel reinforcement).

1.6.3.1 General

a. The size of elastomeric bearings shall be such that the external steel load plate and/or elastomer surfaces are in full contact with the loaded surface under all loading conditions.

b. The properties of elastomeric compounds depend on their constituent elements. Where shear modulus or creep deflection properties are specified or known for the specific elastomer of which the bearings are to be made, they should be used in the design. Otherwise the values used shall be those from the applicable range given in Table 1.6.2 which provide the least favorable results. Values for intermediate hardness may be obtained by interpolation. The hardness grade (durometer) of the elastomer shall be selected on the basis of the requirements of Section 2.5 of this Chapter and the environmental conditions anticipated at the bridge site. The shear modulus shall be determined using the test specified in Sections 2.5.9 and 2.5.10 of this Chapter. Unless otherwise specified, elastomeric bearings shall be made from a 60 durometer elastomer.

c. If elastomeric bearings are to be used at locations where temperatures less than –32 degrees C (–25 degrees F) can be expected for a period of several days, consideration shall be given to specifying natural rubber and to requiring special testing by the manufacturer for the temperature range expected. The increase in stiffness, embrittlement, and crystallization are areas of importance to be investigated.

1.6.3.2 Loads

a. "Service Load Design" shall be used for the design of elastomeric bearings.

1.6.3.3 Notations

A = Plan area of bearing - mm2 (in2)

a_L = Relative rotation of top and bottom surfaces of bearing about an axis perpendicular to the longitudinal axis of the bridge - radians

a_W = Relative rotation of top and bottom surfaces of bearing about an axis parallel to the longitudinal axis of the bridge - radians

D = Gross diameter of reinforcement of a circular bearing—mm (in.)

d_c = Instantaneous compressive deflection of bearing—mm (in.)

d_s = Shear deflection of bearing—mm (in.)

e_{ci} = Compressive strain of an individual elastomer layer {change in thickness divided by unstressed thickness}

f_a = Average compressive stress on bearing caused by dead load and live load with normal impact—MPa (psi).

= P/A – MPa (psi)

F_s = Shear force on bearing—N (lbs)

F_y = Yield Point of internal steel reinforcement—MPa (psi)

G = Shear modulus of elastomer at the design temperature—MPa (psi)

Modifying factor

k = 1.0 for internal layers of reinforced bearings

= 1.4 for cover layers

= 1.8 for plain bearings

L = Length of a rectangular bearing parallel to the longitudinal axis of the bridge. For reinforced bearings these values refer to the internal reinforcement dimensions—mm (in).

Table 1.6.2

Material Property	ASTM Standard	Test Requirements	Natural Rubber			Neoprene		
Physical Properties	D2240	Hardness (±5)—Shore A Pts	50	60	70	50	60	70
	D412	Minimum Tensile Strength in MPa (psi)	15.5 (2250)	15.5 (2250)	15.5 (2250)	15.5 (2250)	15.5 (2250)	15.5 (2250)
		Minimum Ultimate Elongation—% *	450	400	300	400	350	300
Heat Aging	D573	Specified Temperature in Degrees C (F) {22hrs}	70 (158)	70 (158)	70 (158)	100 (212)	100 (212)	100 (212)
		Aging Time-hrs	70	70	70	70	70	70
		Max change in Durometer Shore A Pts.	+10	+10	+10	+15	+15	+15
		Max change in Tensile Strength—%	–25	–25	–25	–15	–15	–15
		Max change in Ultimate Elongation—%	–25	–25	–25	–40	–40	–40
Compression Strain	D575 Method B	Vertical load in MPa (psi)	7.0 (1000)	7.0 (1000)	7.0 (1000)	7.0 (1000)	7.0 (1000)	7.0 (1000)
		Max Permissible Strain %	7.0	7.0	7.0	7.0	7.0	7.0
Compression Set	D395 Method B	Specified Temperature in Degrees C (F) {22hrs}	70 (158)	70 (158)	70 (158)	100 (212)	100 (212)	100 (212)
		Max Permissible Set %	25	25	25	35	35	35
Optional Requirements								
Ozone Resistance	D1149	Partial Pressure of Ozone in MPa (psi)	50.0 (7250)	50.0 (7250)	50.0 (7250)	50.0 (7250)	50.0 (7250)	50.0 (7250)
		Duration of Test in Hrs.	100	100	100	100	100	100
		Tested at 20% Strain 38° C (100° F) mounting procedure ASTM D518 Procedure A.	no cracks	no cracks	no cracks	no cracks	no cracks	no cracks
Shear Modulus	None***	Modulus at 23° C (73° F) in MPa (psi)	0.7 ± 0.1 (95 ± 15)	1.0 ± 0.1 (140 ± 20)	1.4 ± 0.3 (205 ± 40)	0.7 ± 0.1 (95 ± 15)	1.0 ±0.1 (140 ± 20)	1.4 ± 0.3 (205 ± 40)
Low Temperature Properties**	D2137	Low Temperature Brittleness	no failure	no failure	no failure	no failure	no failure	no failure
	D1415	Low Temperature Stiffness Max change in hardness Shore A Points	+15	+15	+15	+15	+15	+15
	D1229	Low Temperature Compression Set Max %	65	65	65	65	65	65
Tear Resistance	D624	Die C, Min. N/mm (lbs/in.)	32 (180)	32 (180)	32 (180)	32 (180)	32 (180)	32 (180)

*Compounds of nominal hardness between the given values shall have the requirement determined by interpolation between the given values.
**Engineer to specify the required test temperature.
***The test method set forth in Sections 2.5.9 and 2.5.10 of this Chapter shall be used.

P = Vertical load on the bearing—N (lbs)

S = Shape factor of one layer of a bearing

$$= \frac{\text{loaded area}}{\text{effective area free to bulge}}$$

$$= \frac{LW}{2\,t_i\,(L+W)} \qquad \text{Rectangular Bearing}$$

$$= \frac{D}{4\,t_i} \qquad \text{Circular Bearing}$$

T = Total elastomer thickness of bearing - mm (in.) = Σt_i

t_i = Actual elastomer thickness between reinforcing plates of an individual elastomer layer—mm (in.)

t_s = Thickness of one internal steel reinforcement - mm (in.)

W = Width of a rectangular bearing perpendicular to the longitudinal axis of the bridge. For reinforced bearings these values refer to the internal reinforcement dimensions—mm (in).

1.6.3.4 Compressive Stress

a. For bearings which may experience shear deformation, the average compressive stress, f_a, shall not exceed GS/k, nor shall it exceed 7 MPa (1000 psi) for reinforced bearings, or 5.5 MPa (800 psi) for plain bearings. In bearings containing layers of different thicknesses, the value of S/k shall be taken as the smallest value obtained for the various layers of the bearing. Allowable compressive stress may be increased by 10% where shear translation is prevented or a positive slip apparatus is provided. Specifications for a positive slip apparatus are not covered by this specification. Design of a positive slip apparatus must be approved by the Engineer.

1.6.3.5 Compressive Deflection

a. Compressive deflection, dc, of the bearing shall be so limited as to insure the serviceability of the bridge.

b. Instantaneous deflection shall be calculated as $d_c = \Sigma e_{ci}\, t_i$ and shall be less than 0.07T or 3 (0.125 in.), whichever is lower.

c. Values for e_{ci} shall be obtained from design aids based on tests such as presented in Figure 1.6.3A, by testing, or by rational analysis.

d. The effects of creep of the elastomer shall be added to the instantaneous deflection when considering long-term dead load deflections. They shall be computed from information relevant to the elastomer compound used, if it is available; if not, the values of 25% for 50 durometer elastomer, 35% for 60 durometer elastomer, and 45% for 70 durometer elastomer may be used. The 3 (0.125 in.) limit in (b) does not apply for long-term dead load deflections.

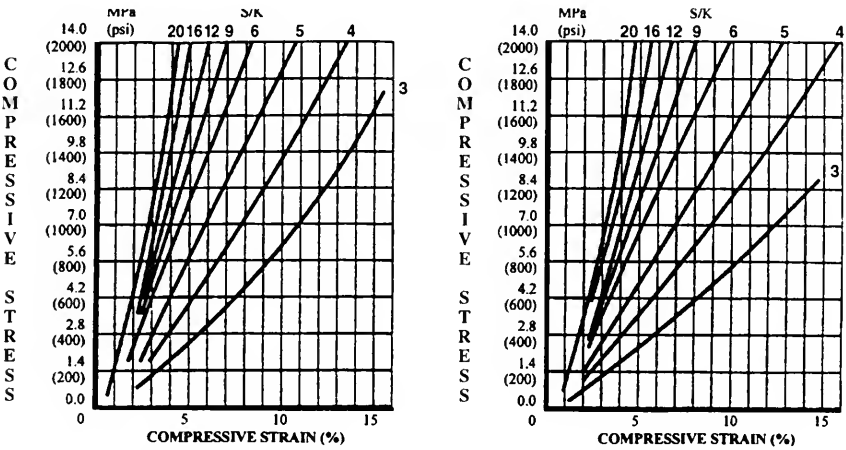
e. The total compressive instantaneous deflection—excluding dead load deflection—of all elastomeric pad and bearing elements that occur between the top of deck and rigid support foundations (pads between deck slabs and girder top flanges, elastomeric bearings, elastomeric pads between masonry plates and top of masonry, etc.) shall also be calculated as $d_c = \Sigma e_{ci}\, t_i$ and shall be less than 3 (0.125 in.).

1.6.3.6 Rotation

a. The relative rotation between top and bottom surfaces of the bearing shall be limited by

$L(a_L) + W(a_W) \leq 2(d_c)$ for rectangular bearings

$D[(a_L)^2 + (a_W)^2]^{1/2} \leq 2(d_c)$ for circular bearings



Compressive Stress/Strain of Steel Reinforced Neoprene Bearings (hardness of neoprene compound—60 Durometer A)

Compressive Stress/Strain of Steel Reinforced Neoprene Bearings (hardness of neoprene compound—50 Durometer A)

Figure 1.6.3A

1.6.3.7 Shear

a. The shear deformation shall be taken as the maximum possible deformation caused by creep, shrinkage, post-tensioning, live load rotation and thermal effects computed between the installation temperature and the least favorable extreme temperature, unless a positive slip apparatus is installed.

b. The bearing shall be designed so that

$T \geq 2\ ds$

c. The shear force induced by shear deformation is approximated by

$F_s = G\ d_s\ A/T$

d. Variations of G with temperature shall be taken into account. Test data from the manufacturer or from special testing for the project, shall be used for the design. Since the physical data can be expected to vary widely, maximum values should be used for obtaining forces involved, and minimum values used to determine shear deflection. Design aids are given in Figure 1.6.3B, and can be used if special project values are not available.

1.6.3.8 Stability

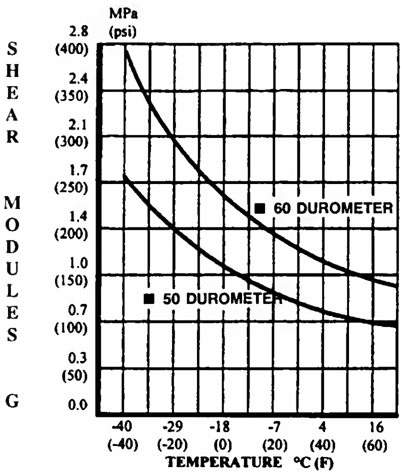
a. To ensure stability, the total thickness of the bearing shall not exceed the smallest of:

L/5, W/5 or D/6 for plain bearings

L/3, W/3 or D/4 for reinforced bearings

1.6.3.9 Steel Reinforcement

a. The reinforcement must be adequate to maintain proper alignment during fabrication and to sustain the tensile stresses induced by compression of the bearing. The minimum thickness is limited to require $t_s \geq 0.092\ t_r$.



Relationship of Shear Modulus to Hardness of Neoprene Compounds at Various Temperatures

Note: Data for graphs obtained from published tests by E. I. Dupont, Inc. and values will vary for other formulations for the neoprene.

• Shore A points hardness determined by ASTM D2240

Figure 1.6.3B

b. For these purposes, t_i shall be taken as the mean thickness of the two layers of elastomer bonded to the reinforcement. The determination of the steel reinforcement thickness shall take into account an allowance for stress concentration caused by holes in the bearing. Holes are discouraged for all bearings.

1.6.3.10 Anchorage

a. When some combination of loads exists which causes a shear force greater than ⅕ of the simultaneously occurring compressive force, the bearing shall be secured against horizontal movement. When the bearing is attached to both top and bottom surfaces, the attachment must be such that no tension is possible in the vertical direction. When the dead load stress on the bearing is less than 1.4 MPa (200 psi), or the horizontal loads are greater than the frictional resistance when using a coefficient of friction of 0.20, the bearing shall be restrained against horizontal movement.

1.6.3.11 Stiffeners for Steel Girders

a. Steel girders seated on elastomeric bearings must have flanges which are stiff enough locally not to cause damage to the bearing. Any necessary stiffening may be accomplished by attaching a sole plate to the bottom flange of the girder or by vertical stiffeners connected to the girder web and flanges. The requirements of AREA Manual Chapter 15 shall govern the design of steel girder stiffeners and connections.

b. Single-webbed girders symmetrical about their vertical axis and placed symmetrically on the bearing need no additional stiffening if

$$b_f/2t_f \leq [F_y/3.4\ f_s]^{1/2}$$

where

b_f = total flange width—mm (in.)

t_f = flange thickness or combined coverplate + flange thickness—mm (in.)

f_y = yield point stress of girder steel—MPa (psi)

c. If the requirement of Paragraph 1.6.3.11(b) is not satisfied, it will be necessary to add two or more bearing stiffeners on each side of the girder at a spacing "a" given by the following requirement:

$$a \leq [(t_f)2\ F_y\ /1.2\ f_a]^{1/2}$$

where

F_y = yield point stress of flange steel.

1.6.3.12 Geometrics

a. Misalignment in bridge girders due to fabrication tolerance, camber, or other source, shall be considered in the bearing design or shall be accounted for with tapered sole plates or by a device which prevents eccentric loading on the bearing.

b. Bearings which are used in pairs shall be placed along an axis perpendicular to the longitudinal axis of a beam.

1.6.3.13 Alternate Design Procedures

a. The design of bearings by procedures other than those outlined above shall be permitted, at the discretion of the Engineer. Such procedures shall take into account the stresses and deformation in the bearing determined from a rational analysis and the design shall be based on the material properties pertinent to the elastomer of which the bearing is to be made. Performance shall be verified by test using the standards of Level II certification given in Section 2.5, and in addition, the effects of instability and fatigue shall be investigated.

1.7 MULTI-ROTATIONAL BEARINGS

(UNDER DEVELOPMENT)

1.8 SEISMIC ISOLATION BEARINGS AND DEVICES

(UNDER DEVELOPMENT)

Part 2

Construction

1997

FOREWORD

The purpose of this part is to formulate specific and detailed requirements for the construction of bearings for nonmovable railway bridges.

CONTENTS

(Under Development)

Part 3

Commentary and Bibliography

1997

FOREWORD

The purpose of this part is to furnish the technical explanation of various articles in Parts 1 and 2 and to furnish supplemental recommendations for use in special conditions. In the numbering of articles of this part, the second and succeeding digits in each article represent the article being explained.

CONTENTS

3.1 DESIGN (Under Development)

3.1.2 Basic Allowable Stresses

The allowable stress in bearing between rockers and rocker pins was adapted from editions of AREA Manual Chapter 15, Article 1.4, prior to the 1969 edition and the low value of 0.375 Fy was retained to minimize pin wear. Pin wear had historically been a cause of trouble when higher values for this condition were permitted.

3.1.3 Steel Bearing Components

3.1.3.3 Shoes and Pedestals

The requirements of Article 1.3.3 provide that the load is uniformly distributed over the entire bearing surface, and that, in the case of welded bearings, the load is transmitted in bearing.

3.2 CONSTRUCTION (Under Development)

BIBLIOGRAPHY

The Bibliography includes only the specific material used in development or explaining specification requirements. In most cases, these requirements are supported by studies and tests reported in other engineering literature.

(Under Development)

TWO STEEL TRUSS REPLACEMENT PROJECTS ON THE SANTA FE

By: D. E. Lozano*

Steel truss bridge replacements are always challenging projects. Railroads generally use truss designs for their longer spans over rivers, major highways, or deep canyons. Consequently the spans are always difficult to access, and are usually too heavy to lift. The following presentations entail replacement of two truss spans where innovative techniques were utilized to minimize train delays. Both projects were completed on the Santa Fe Railroad in 1995. Logistics for these two projects were especially complicated because of their locations. One was in a densely populated city; the second over an environmentally sensitive river.

Aviation & Rosecrans Intersection Truss Replacement

The first project is in the metropolitan Los Angeles area. The original bridge was constructed in 1924 over a 45 degree skew intersection of Rosecrans and Aviation streets. The 100 ft. through girder structure spanned two traffic lanes in each direction.

Forty years later, urban growth required additional traffic lanes. The 100 ft. girder span was replaced with a 164 ft. through truss span in 1964. The truss was built on an offset "dog leg" alignment, but the original embankment was left undisturbed. The existence of the original embankment played an important role in the 1995 truss replacement as you will see in forthcoming slides.

Thirty years of population growth resulted in increased traffic through the intersection and the bridge again needed to be lengthened to accommodate more traffic lanes. In 1994, a cooperative effort between the City of El Segundo and the City of Hawthorn resulted in funding for a new bridge. Hawthorn City Engineer Charles Herbertson was designated Project Director. A new 300 ft. through truss was designed by De Leuw, Cather & Company to replace the 164 ft. truss. Fletcher General Construction Inc. was successful bidder for the project.

There were major obstacles, both physical and political, which had to be overcome during construction. In 1993, a commuter light rail system was constructed parallel to the freight line. A post-tension cantilevered cast in place concrete structure was built over the intersection in very close proximity to the 1964 steel truss. The commuter rail bridge severely restricted access to the old truss.

The freight railroad had to be continuously operable for eight trains daily. High density automobile traffic precluded street closure Monday though Friday. Even weekend closures were politically volatile. Public safety is affected because the streets are emergency routes for police and fire units. The 405 freeway off-ramp for Rosecrans Street had to be closed concurrent with the intersection closure. The 405 freeway, as well as local surface streets, quickly become a traffic nightmare whenever the intersection is closed.

Normally, a truss is assembled "in-place." This method would require continuous falsework supports under the steel members as they are bolted together. It would be impossible to use this method because the intersection would be blocked by falsework for several months. The truss had to be assembled somewhere else, other than over the intersection, and moved into final position.

Fletcher General Contractors Project Managers John Meagher and Mike Kennedy devised a plan to assemble the new 300 ft. steel truss on the old railroad embankment at one end of the intersection, and then forward launch it into place. The truss assembly would be supported on steel runways placed parallel to the existing railroad track. The old railroad embankment, that was left in place

*Assistant Director Structural Engineering, BNSF Corp.

from the 1924 girder span, was just wide enough to provide working room for the new truss. The new track would revert back to the original tangent alignment that existed when the original girder structure was in service.

The live track was shored for a 27 ft. deep excavation using sheet piles and tie backs. The new abutments were cast without interruption to trains. A petroleum pipeline was discovered where the new footing was to be cast. A loose fit PVC sleeve was placed around the pipeline and the concrete footing poured around the sleeve. The concrete abutments were cast concurrent with steel fabrication at Universal Structural Incorporated in Vancouver, Washington. All fabricated steel was painted at Vancouver before it left the shop.

Figure 1 shows various stages of steel assembly on the old embankment section. Notice the temporary handrails and elaborate system of fall protection devices. Because the truss was to be forward launched, it had to be assembled in alignment with the direction of movement. Assembly tolerance was plus or minus ½ inch in 300 ft.

Assembly time was approximately 6 weeks. The finished truss required 20,000 bolts and weighed 800 tons. It was 21 ft. wide, 55 ft. tall, and 300 ft. long. The old 164 ft. truss appeared quite small compared to the new bridge. The supporting timber blocking under the bottom chord was removed and replaced by eighteen Hilman rollers on a steel channel runway.

A July weekend was selected the forward launch. The intersection was closed Friday evening at 9:00 P.M. Twenty five 200 kip capacity steel falsework towers were carefully positioned across the intersection. The Contractor took extraordinary care to insure that all tower members were on line and perfectly plumb (Figure 2). The crews worked around the clock. Figure 3 shows one of the stages of the launch runway construction. Train traffic continued without interruption throughout the launch. A D-8 Cat and a 9 part reeved block provided power to pull the truss across the runways. A crowd of spectators gathered throughout the day in hopes of watching the truss move across the intersection.

The runway falsework was completed at 2:00 A.M. Sunday morning. A job briefing was conducted just prior to the launch. The Foreman emphasized that they were not to take any chances. There was no margin for error. The truss weighed 800 tons. If the launch went off line or a tower leg buckled, it could be catastrophic. There was no practical way to correct launch direction or, worse yet, pick up the truss if it were to fall.

Figure 1

Figure 2

Figure 3

There was an great apprehension, even from the seasoned iron workers, as the dozer pulled slack from the lines. The first pull moved the truss about 4 inches, accompanied by a plethora of pops, groans and squeals as the rollers, runways and truss members took on loads.

Gradually, as everyone's confidence built up, the movement increments were increased. The truss rolled across the intersection very slowly as all eyes watched falsework towers for signs of unusual deflections. The actual launch began at 2:30 AM Sunday and was nearly completed at 7:00 AM. At daybreak, a tower leg began to buckle just before the truss reached final destination. The Contractor shored the buckled post and continued the launch, finishing longitudinal movement by 12:00 noon on Sunday.

The truss needed to be rolled into position above the top of the concrete abutment parapet wall. The launching runway elevation was significantly higher than the bridge seat elevation. An elaborate hydraulic jacking apparatus was designed by Smith, Monroe & Gray Engineers, Inc. After the truss was in final longitudinal position, it was raised six inches to provide clearance for removal of the runway channel beams. After runway beam removal, the truss was slowly jacked down seven feet onto the bridge bearings. Falsework towers were removed during the jacking process.

The intersection was clear of falsework by Monday afternoon, but the 800 ton truss was still suspended five feet above the bearings. The City Engineer decided to prohibit traffic under the span until the truss was jacked down closer to the bridge seat. The intersection was finally opened Wednesday morning at 6:00 A.M. when the truss had been lowered to within 1 ft. of the bearings. The intersection closure ran two days longer than planned.

Although the street intersection was closed four days, train traffic continued without interruption. The only track window required was to cut over the former "dog leg" alignment to the new tangent track over the 300 ft. truss. Train departure times were rescheduled to provide the necessary window.

Removal of the old truss presented another sizable problem. The old 180 ton truss was landlocked by the new truss on one side, and the concrete light rail bridge on the other side. The truss could not be lowered into the street unless the abutments were removed or the truss shortened.

The Contractor decided to use two cranes to lift the truss off the bearings, and then torch cut about five feet off each end. The cranes would hold the truss suspended while the ends were being cut. After the ends were severed, the remaining truss would be short enough to clear the abutments and could be lowered onto the street. The location of the lifting points required an additional steel tension member to replace the severed bottom chord. Portal bracing was added to take up compressive forces normally resisted by the severed end floorbeam.

The bridge was lightened as much as possible by removal of ballast, steel decking, and most of the deck stingers prior to closing the intersection for truss removal. Final cutting of floor beams and truss ends were done with an ARCAIR SLICE SYSTEM. The cutting tool uses only oxygen forced through a special hollow steel cutting rod. The steel rod self consumes, producing tip temperatures of 10,000 degrees. The system cuts twice as fast as an oxygen-acetylene torch, but consumes a lot of oxygen in the process.

After the end sections were severed, the cranes lowered the truss into the intersection. The end sections remained on the bridge seat, supported on wood blocking. They were later removed by the cranes. The Contractor intended to use a LaBounty 100 ton hydraulic rotating shear mounted on a Cat 245 chassis to cut the truss members. Unfortunately, the old truss proved to be tougher than the shear. The hydraulic shear made some serious dents, but it could not cut through the larger truss members. The Contractor resorted to oxygen-acetylene torches to finish the job.

Figure 4 is a view of the new truss as it looks today.

Figure 4

Stanislaus River Truss Replacement

The second project involved replacement of a 175 ft. truss built in 1905 over the Stanislaus River in the central California community of Riverbank. Eight Amtrak trains and twenty four time sensitive freight trains use the track daily. The river is used by salmon for annual spawning runs. The bridge is adjacent to a City park and there were sensitive environmental issues to mitigate.

The truss was struck by a shifted load in 1993.

Several members damaged by the shifted load required replacement, while others were spliced to provide adequate strength until a new structure could be designed and built. During the design process, a Consultant was hired to secure permits from the U.S. Fish and Wildlife, Army Corps of Engineers, and the California Department of Fish and Game.

The Consultant's biology report indicated that a threatened species, the "Valley Elderberry Longhorn Beetle," might someday make it's home in the Elderberry bushes located under the bridge. Please note that this is a *Threatened Species*; not an *Endangered Species*. If it were "Endangered," the environmental costs would likely be much higher. Also please be aware that a biological survey did not locate any of the threatened species of beetle in the Elderberry bushes. In fact, there was no sign of any active beetle species, threatened or otherwise, within the last five years.

The Elderberry bush itself is not endangered or threatened. But because the bush is a potential host for a threatened species of beetle, we were required to transplant 111 elderberry bushes. Cost of environmental consultants, biologists, permits, and transplant costs totaled $960,000. That calculates to about $8600 per bush.

The environmental costs included a permit from the State Historical Preservation Officer. They surmised that the bridge was eligible for listing in the National Register of Historic Places. Documents had to be filed to demonstrate to the Historical Preservation Officer that efforts were made to advertise and market the truss to anyone who may be interested in "rehabilitation and preservation" of the span. Two consecutive weeks of advertising in local papers produced not one single buyer. Therefore a permit was issued to allow Santa Fe to demolish the truss.

The replacement structure, consists of two beam spans and a new pier. The span lengths are 50 ft. and 127 ft. The non-symmetric span lengths were designed to locate the new pier as close as possible to the bank of the river to minimize environmental impact to salmon migration. The new pier was constructed at the waters edge, surrounded by a sheet pile cofferdam. Foundation piles were driven, followed by forming and pouring the pier footing and stem. The existing piers were modified with raising blocks to accommodate the new beam spans. Cost of pier construction and modifications was $748,000.

Concurrent with pier construction, the steel beams were fabricated using A-588 weathering steel. The beams were trucked into the construction site. The beam spans were assembled on the ground complete with track panels, walkways, handrails, and fall protection systems. Two of Santa Fe's most seasoned bridge supervisors, Mike Rand and Max Tenario, were assigned to build the beam spans and coordinate Contractor activity.

The project required that the 334 ton truss be removed and the new beam spans set in place with minimal train delay. Although Amtrak passengers can be bussed, and freight trains could be detoured over our competitors tracks, neither was desirable. The old span could not be demolished over the river because it was coated with lead base paint.

After consulting with Bragg Crane Company, we opted to lift out the truss in one piece utilizing a Manitowac M1200 Ringer crane. This awe-inspiring 1500 ton capacity machine is assembled at the job site by a crew of seven using a separate 250 ton crane. The boom is 225 ft. long. The counterweights are hollow shells that are filled with gravel at the assembly site. The counterweights are stacked 40 ft. high. The ring foundation and timber blocking are 60 ft. in diameter. The assembled crane weighed 3.5 million pounds. Fifty six semi trucks are required to import the crane components to the job site. It took two weeks to assemble the crane (Figure 5).

The hook block is fifteen ft. tall and weighs 21 tons. Fortunately, the slings are made of light weight Kevlar materials which made rigging a little easier.

Figure 5

The lifting point locations for the old truss was a complicated design process. Lifting eyes were attached to a pin joint in the top chord. High strength stands extended from the top pin joint to a bottom chord pin. The tension strands were necessary to redistribute forces so that the truss could be lifted by rigging attached to the top chord. A threaded connecting apparatus allowed for adjustment of prestressing forces in the strands. Santa Fe Bridge Engineer Victor Tamosiunas designed the lifting mechanism, which worked quite well.

Word of the project spread through the surrounding rural communities. On the day of the change out, crowds gathered in the adjacent park. Santa Fe Supervisor Max Tenario described it as a carnival atmosphere; people were setting up lawn chairs and picnic baskets. Vendors sold steak sandwiches and drinks. They came on horseback, boats and automobiles; lots of automobiles. Streets were so congested that twenty five Police, Sheriff, and Highway Patrol officers were dispatched to direct traffic. Television stations sent field broadcast units to cover the story. It was front page news for several local newspapers.

A Sunday evening was selected for the change out. Arrangements were made for a 24 hour track window. Santa Fe freight trains were rescheduled. Busses were lined up for the morning Amtrak runs. The track window was granted at 6:00 P.M. after the passage of a high priority intermodal freight train.

A job briefing was conducted on the bridge. A 988 loader removed the track panels, lifting them from one end and backing off the bridge. Ballast was removed to reduce the lifting weight. Bearing anchor bolts were torch cut. It was well past sunset when the crane operator received instructions by radio to begin the lift. The lift was made very slowly. The truss was raised initially only two inches and held suspended to allow the load to equalize. As the truss was held in suspension, strand tension was adjusted.

Figure 6

The truss was then lifted above the top of rail elevation. Radius was increased and the truss lowered to just above the water. The crane was rotated to position the truss over the river bank, and set down on blocking. The entire process to remove the truss, including removal of track and ballast, took 2 hours.

The short 50 ft. beam span, weighing 59 tons, was then lifted into place. The longer 127 ft. beam span, weighing 235 tons, was set last. Both spans were in place and bearings anchored by sunrise. The old span was resting on the bank where it could be easily demolished without fear of getting lead paint debris into the river.

The track department bolted together track panels, using six inch cinder blocks under the ties as shims. Ballast was center and side dumped and the track was completely surfaced and ready for the first train by 11:00 A.M. Total elapsed time for all bridge and track work was only 18 hours. The beam spans eliminated a clearance restriction that existed since 1905 (Figure 6).

The Manitowac crane was key to accomplishing the work within the constraints of the environmental issues. Move-in/move-out cost was $570,000. Operation cost is $1,150 per hour. Total crane cost was $770,000.

The total cost of bridge replacement was approximately $3,000,000, or about $17,000 per track foot. This is double the average cost per track foot for similar projects. The additional costs were, for the most part, attributable to environmental mitigation requirements.

Acknowledgements

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Jack Wittmeyer—Santa Fe (retired), Charles Herbertson - City of Hawthorn (310) 970-7955,

Virgil Popescue—L.A. County Public Works (818) 458-3122, Mike Kennedy - Fletcher General Contractors (714) 553-8800, Dave Marques - Bragg Crane (916) 834-1605, Smith, Monroe, & Grey Engineers, Inc. (503) 643-8610

Jeff Stapleton of De Leuw, Cather & Co. (303) 837-4007, Woodcrest Engineering (shoring) (909) 780-2843

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USAGE OF PROJECT MANAGEMENT TOOLS

By: Bruce R. Pohlot*

I have been requested to focus my presentation on tools that can assist in the management of our project work. "Management" is the key word.

There are many individuals and entities, inclusive in our own Railroad organizations, that view Engineers as stated by one Vice President of Transportation as cube dwellers, not Managers, but people who sit at their computers and calculators solving problems with no integration of their work into the big picture. They are wrong. There are many successful Engineers in the business world. To name a few, Jack Welch at General Electric, W.L. Fuller, Chairman of the Board of Amoco, and Lee Iacocca, a graduate from one of my Alma Maters, who brought Chrysler out of bankruptcy.

We as Engineers must be business people in the 90's—Entrepreneurs in our own environment. As we pass thru a period of unprecedented change, areas that we thought we knew are now unknowns. Our organizations are in constant flux, downsizing and . . . reengineering. Increase productivity with reduced resources is a common theme we all are facing. So, how are we going to continue making advancements while reducing costs? I believe the answer lies in Project and Program Management.

Our success is dependent on planning, direction, scheduling, monitoring, and control. These project functions must be closely tied together by an Information system if the performance is to be adequately measured and controlled. For efficient project operations, a single Information and Control System should be used, not separate project and functional department cost control systems. (1)

Remember the old saying, "You can't manage what you can't measure". Tools and techniques must look at the whole picture, not just end results or productivity rate or schedule compliance. We must see the entire program as a collection of activities, where each individual project is subservient to the whole. Otherwise, we will "be like the man whose only tool is a hammer and where every problem looks like a nail".

Approximately a year and a half ago Amtrak realized the needs and benefits of creating a Program Management organization within the Engineering Department. The term "Project Management" and "Program Management" are frequently used interchangeably. However, they have different meanings. At Amtrak, Project Management is a function that coordinates multiple activities or even multiple projects primarily for the purpose of optimizing the use of common resources within a functional area. These individual projects, although part of some overall strategic plan, are managed as an entity of itself.

In the Railroad Industry, these projects reside in our production, construction and maintenance functions. Program Management, on the other hand, is a collage of multiple projects aimed at the long term strategic objectives of the corporation.

A program such as Amtrak's Northeast Corridor Improvement Program can have durations that extend for decades.

Amtrak's Program Management organization was established to create synergy through the coordination of functions and activities by interfacing at all levels of the Engineering organization in a "participative management style". A "soft matrix" organization was developed that keeps the Project Management responsibilities within the functional areas but allows a cross-functional integration that ties in the various elements so that the individual projects are performing in unison toward an overall objective. (Exhibit 1)

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*AVP Engineering—Program Management, AMTRAK

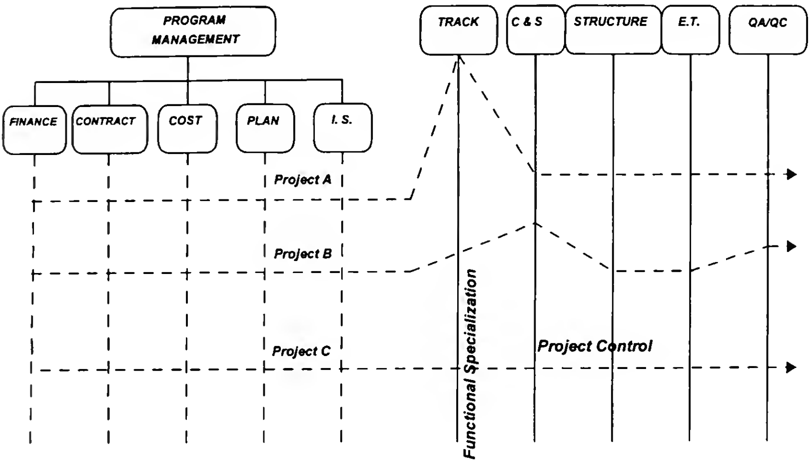


Exhibit 1. Matrix Flow Chart

Prior to the creation of Program Management, projects were housed solely in the functional area with integration occurring only at a high level. Actually, projects did not come together until they reached the Vice President-Chief Engineer. Now Program Management acts as the "lynch pin" tying the functional area together in support of strategic goals and objectives that use the same resources and is ultimately responsible for the effectiveness of the program. Program Management acts as the "agent of the owner" where the functional areas of the Engineering department are the owners and all activities are reviewed in the best interest of the corporation.

With full knowledge that a soft matrix organization is the hardest to make work, we had to establish a methodology that would foster vertical, horizontal and diagonal communications. Thus, we began the implementation of Concentric Program Management. This can be viewed as the CTC of the Engineering Department. (Exhibit 2)

Program Management pulls together the required data from various elements in order that a dynamic information flow is available to all levels of Management. Our Automatic Labor Collection System (ALCS) is comprised of hand held computers that are utilized by foreman to enter time and production. Presently, we are in field testing with full implementation expected by this summer. The results will be real time labor information and production reporting. Our corporate Financial Information System (FIS) is also accessed for verification of charges and receivables information. Within Program Management, an integrated project analysis is performed on cost and schedule. Project simulations are run to resolve conflicts among projects and on potential delays due to train traffic. Also, "What if" analyses are performed on schedule and costs to find the optimal critical path of the program. The Project Management section is comprised of all the functional areas, the Divisions, as well as the projectized organizations. (Exhibit 3) Their planning and scheduling data, when approved, is uploaded into the master schedule. This allows sub master planning and scheduling to be integrated with the overall program. This two way information flow results in the optimization of resources, particularly track outages, so that we can approach our transportation colleagues with an overall detailed occupancy requirement with impacts if changes are requested.

To break down Concentric Program Management one step further, let's look at planning and scheduling within a functional area. (Exhibit 4) At this level the authorized projects are dissected into

- Labor Collections System
- Financial Database
- Master Schedule
- Scheduling Tool
- Cost/Schedule Performance
- Risk Analysis
- Cost Estimates

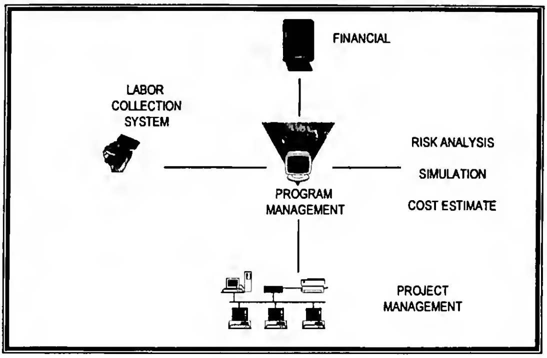


Exhibit 2. Concentric Program Management

- TRACK DEPT.
- ET DEPT.
- C&S DEPT.
- NECIP/NHRIP
- STRUCTURES
- DIVISIONS
- PROGRAM MANAGEMENT
- NY ZONE

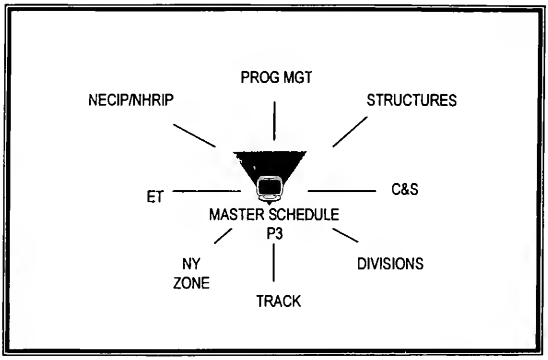


Exhibit 3. Integrated Departmental Planning

- TRACK DEPT.
- ET DEPT.
- C&S DEPT.
- NECIP/NHRIP
- STRUCTURES
- DIVISIONS
- PROG. MGT
- NY DIVISION

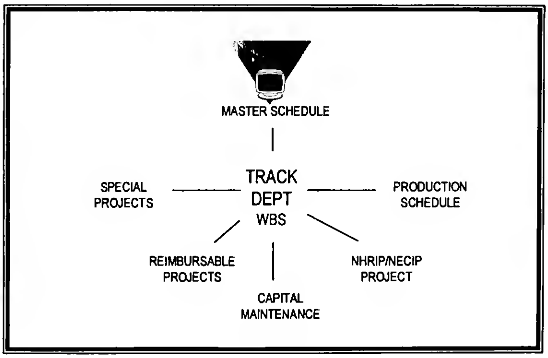


Exhibit 4. Project Level Planning Within Department

Work Breakdown Structures (WBS) and their detailed schedules are worked within the framework of the approved master schedule. At this level they can arrange and change the internal planning to perform "what if's" and view any conflicts to other projects. The project breakdown into task levels through a work breakdown structure not only enhances the planning process but will depict manpower requirements including the support functions. Eventually, this will allow the leveling of manpower, smoothing out the big peaks and valleys.

To provide you a brief overview of the magnitude of our program, Exhibit 5 depicts the funded dollars by geographic area, and the daily number of train movements. It becomes obvious that without prior, proper scheduling of the work activities, either two things happen. Trains are delayed, or cost overruns occur. Thus, associated resources, whether they be track outage, manpower or equipment must all be inputted into WBS and uploaded into the master plan so overall resource requirements can be reviewed, manipulated and scheduled.

In order to show the complexity of the program process, here is the train activity stringline for New York to Philadelphia, PA. The time frame shown is 4 hours and as you can see the window of opportunity, without affecting train movement, is very small. However, with detailed planning, we are able to get "buy in" from our Transportation Department, the Divisions and the associated Commuter agencies.

The strength of good Program Management is not in the ability of a particular software tool, but it is the understanding of Project Management practices. There are two particular project evaluation processes that I personally feel are paramount in Program Management success. They are the Work Breakdown Structure (WBS) and Earned Value. (EV)

The WBS is basic planning. It is the foundation for Project Management, acting as a vehicle for breaking the work down into manageable and definable work packages, thus providing a greater probability that every major and minor activity will be accounted for. Remember the old saying "you cannot eat an elephant in one bite". You must cut it up into small chewable parts. Similarly, the program is divided into projects, projects into work packages and work packages into work activities and so on. Each level of the WBS is comprised of:

- when and where the work activity will be performed
- the scope of work of the particular element

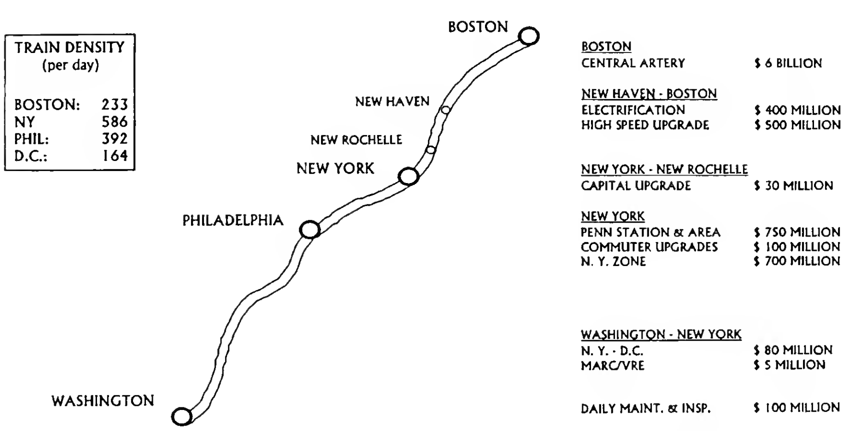


Exhibit 5. Funded Project Construction in Northeast Corrodor

- the time duration
- what resources are required.
- the cost of the activity categorized by labor, material, equipment, and other
- who owns responsibility for the activity.

The requirement for a work breakdown structure will insist that the responsible person, whether he or she be a Supervisor, Project Engineer or Project Manager, provide us with a well thought out plan for each work package. The reason I say that the use of WBS will "insist" on a quality plan is that accurate and relevant status reporting, via earned value techniques, which I will address shortly, will quickly reflect a bad estimate or lack of proper planning.

Managements' fundamental job is to affect outcomes. To affect outcomes, it is necessary to compare the results of effort against expectations. This must happen in a timeframe that allows recognition of a need for corrective action. For these reasons, Amtrak is now adopting the E.V. concept.

To regress for a moment, earned value concepts have been used in the Department of Defense (DOD) since 1965. Over the years private industry has attempted to adopt the E.V. concept, however, they have found the principles of DOD Costs/Schedule Control System Criteria (CS^2) to be too cumbersome.

Basically, too many rules, too much formality with a whole new set of vocabulary. However, with the reissuance of the DOD criteria in 1991, there is a renewed interest in earned value management techniques, not only by other government agencies but also by other countries and the private sector (2).

A striking example of earned value use in the business community is Motorola's 3 billion dollar IRIDIUM project. In consideration that employee rewards are based on E.V. indices of Cumulative Cost Performance Index (CPI), and on the Schedule Performance Index (SPI), one must conclude that they place a high value in these tools. I quote "the best evidence of the commitment of the program to using earned value is the fact that the Motorola satellite communications division employee bonus program is based in a large part on SPI and CPI in 1994". (3) It must be noted that Motorola's use of E.V. would not be considered by a government review team to be in compliance with CS^2, but it's working. It has been an effective application of the concept and probably agrees with the intent of CS^2 founders.

As stated, we are implementing E.V. techniques and preliminary feedback is reassuring. Like Motorola we are not compliant with the entire range of DOD rules, but we are following many of the concepts of the Project Management Institute and the intent of the criteria. The requirements to implement E.V. reporting are just good Project Management techniques, such as those shown on Exhibit 6. E.V. is a simple cost measurement technique that portrays the true cost of doing work and a mathematical way of showing the schedule.

E.V. is most valuable in forecasting the final results of any project. E.V. is a 'strategic' trend indicator. It adds a third dimension to traditional cost management measurement. That is, E.V. or the planned cost times the percent complete. It answers the questions—

- If we stay on course where will we end up?
- How much money will it cost to complete the project?
- How long will it take to get there?

Traditionally, we viewed project status by comparing actual work that was performed to the estimated cost. (Exhibit 7) This method leaves these questions unanswered:

- Are we on schedule and overbudget?

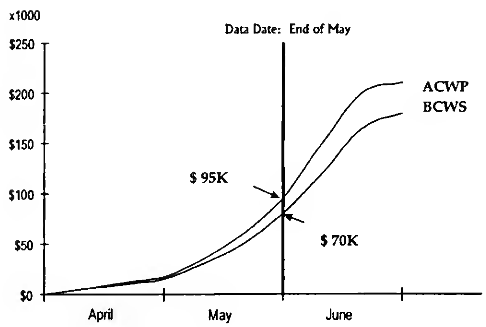
Or

- Are we ahead of schedule and on budget?

- **MANAGEMENT COMMITMENT**
- **SCOPE THE PROGRAM WITH WORK BREAKDOWN STRUCTURE**
- **PLAN & SCHEDULE PROJECT SCOPE**
- **ASSIGN BUDGET TO TASKS**
- **DEVELOP AND MONITOR PROJECT BASELINE**
- **TREND ANALYSIS ON CUMULATIVE CPI & SPI VALUES**

Aggressive management actions are taken when project forecasts are higher than defined budget...

Exhibit 6. Earned Value Techniques



- Optimistic manager might say, *"we are ahead of schedule and on budget".*
- Cautious manager might say, *"we are behind schedule and over the budget".*

Exhibit 7. Traditional: Planned vs. Actual Cost

Utilizing E.V. techniques a $30,000 cost overrun is verified (Exhibit 8). Although the project is still unfavorable, the Project Manager knows that corrective action is warranted.

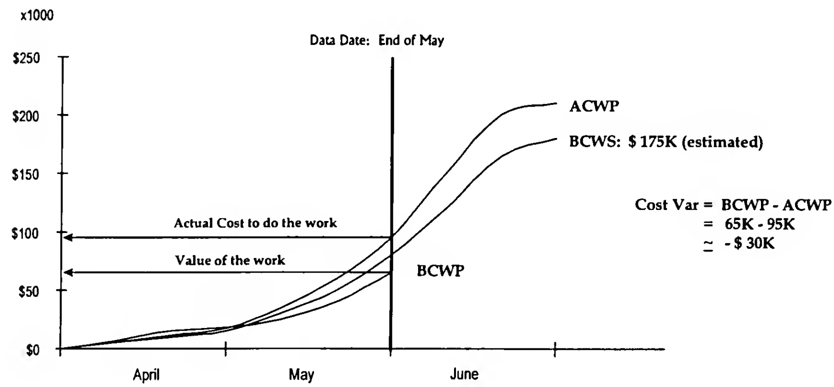
Review of the schedule status shows a negative variance and therefore also behind schedule. (Exhibit 9) In simplistic terms, the level of effort has produced less and at higher costs.

Again, I must reiterate that the validity of these indices are subjected to many variables. The two main variables being—

1. The quality of the WBS. Remember the saying "Garbage in—Garbage out".

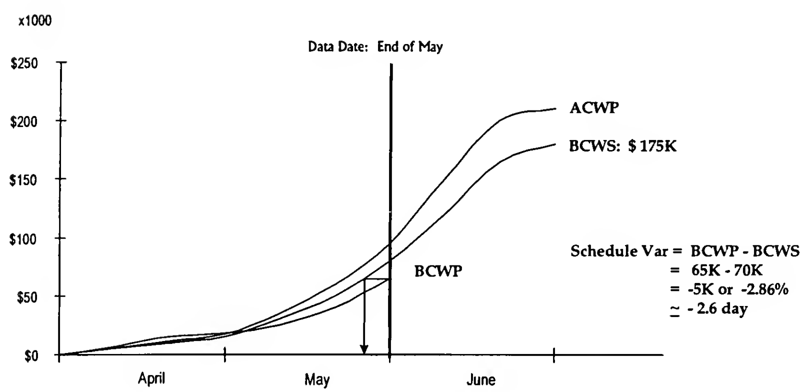
2. Actual performance data. The financial information being up to date and the percentage complete based on real measurements.

Projects using the E.V. concept allows performance measurement against two standards. The planned schedule and cost efficiencies. In today's business environment where an Executive's time is a scarce resource, there is a need for indicators that quickly depict an exception to what is expected. E.V. allows the "management by exception" (3) in the execution of the project plan. Indices that I feel are most relevant to this kind of management are the cumulative CPI and SPI. At this time, it would be appropriate to mention how SPI and CPI are numerically quantified. Perfect performance



Negative cost variance indicates over spending

Exhibit 8. Earned Value: Cost Variance



Schedule Status:
≃ 2.6 days behind planned Schedule
Not all the work was accomplished as planned

Exhibit 9. Earned Value: Schedule Variance

of either index, for an earned value project is 1.0. If either the SPI or CPI falls below 1.0, it should be a wake up call to the Manager that something is wrong. The project team should want to know why and then take corrective action to improve performance on the remaining tasks.

In my opinion, cumulative CPI is the most valuable index. It indicates the spending efficiency by measuring the value of work accomplished as a percentage of actual costs, or to state it differently, the E.V. divided by actual cost of work performed.

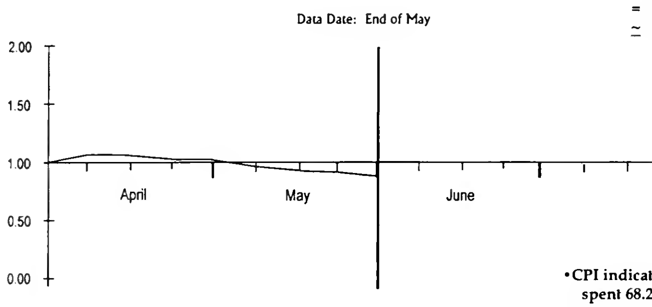
I prefer the use of cumulative CPI over monthly incremental data. Monthly data is too prone to wide fluctuations caused by the planned or actual cost being in the wrong place. Cumulative CPI tends to smooth out these variances. Studies have shown that cumulative CPI does not change by more than 10% once a project is 20% complete. In fact, in most cases cumulative CPI only gets worse as a project proceeds (4). Therefore, if you have 10% overrun when the project is a quarter finished, you will most likely complete the project with at least a 10% overrun.

Let me use a hypothetical example to explain Exhibit 10. Suppose a gang is scheduled to install 1 track mile of CWR per day for a cost of \$100,000 dollars. They actually installed 3,500 feet for \$100,000. Therefore, their CPI to date is .68 or for each dollar spent 68 cents of work was performed.

The Schedule Performance Index - SPI is a representation of how much of the original scheduled work has been accomplished. In other words, did the project accomplish what it set out to do in the same timeframe?

Exhibit 11 depicts that the project is close to being on schedule. However, remember the CPI. It is costing more than estimated to get the job accomplished. Very possibly unfavorable costs are a result of the Project Manager pushing too hard to keep the project on schedule without paying proper attention to the associated costs. It is noteworthy to mention that the SPI, if all of the scope of work is completed, will at the end of the project equal 1.0. Therefore, I also recommend using critical path performance as the third indicator. The E.V. SPI when compared to the CPM provides the project team with a true schedule position of the project, a means to accurately forecast how long the project will take to complete. (3)

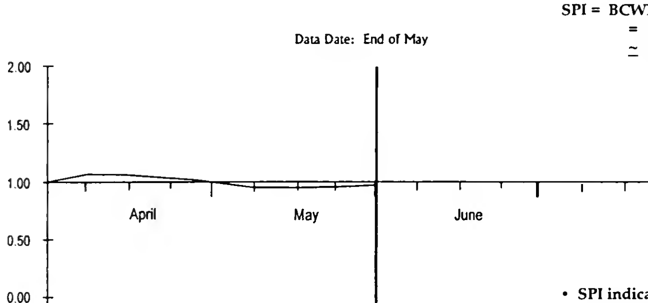
CPI = BCWP/ACWP
= 65K/95K
≃ .682



•CPI indicates for every dollar spent 68.2 cents worth of work was accomplished.

Exhibit 10. Earned Value: Cumulative Cost Performance Index (CPI)

SPI = BCWP/BCWS
= 65K/70K
≃ 0.9286



• SPI indicates 92.9% of the planned work has been accomplished.

Exhibit 11. Earned Value: Cumulative SchedulePerformance Index (SPI)

The use of SPI, CPI and CPM in concert eliminates the battle cry of the Construction Manager "We're behind schedule", "Everyone on overtime", or "I need more people, I'm behind schedule". Either way, this kind of reaction by the Manager will mean that the project will be absorbing more resources to do the same work. It improves the SPI, but will cause irrevocable damage to the CPI. Remember my previous statement, the SPI will eventually correct itself at completion, but the monies spent that cause an overrun shown in the CPI will not be recovered.

At Amtrak, we are implementing the tools and techniques that have been stated in this presentation. Exhibit 12 depicts our 1996 Major Project Program. Each small bar represents a project or work package and provides information by location, type of project, duration, track outage requirement and the critical path. This schedule has proven to be extremely useful to our Transportation Department during our pre-construction planning effort. In consideration that a track outage in New York impacts the availability of an additional outage in Maryland, having a plan that looks at the whole picture has been welcomed with opened arms.

Exhibit 13 is one of the projects from Exhibit 12, shown at the work package level. Not shown is that each bar is additionally broken down to at least 10 more activities or elements. The Histogram depicts costs for labor, material, equipment, contractors and engineering design. To the left of the data point, that is the darker vertical line which extends through the CPM, the Histogram reflects actual cost. To the right of the line are projected or estimated costs. The S curves are:

- Green—BCWS—Our estimate for the project work (top line).
- Red—ACWP—This reflects actual cost up to the data date (second line).

Then to the right of the data date, it estimates cost for the completion of the project.

- Yellow—BCWP—E.V. which stops at the data date (third line).

This project status report allows the project manager to review his E.V. data with the critical path and actual cost, thereby providing a true perspective of the projects overall status. As you can see, the CPI is .93 and the SPI is .87, therefore, the project is running slightly behind. However, with management involvement, this project can be completed within budget.

It has become apparent that proficiency in the use of project management tools makes good business sense. The WBS in of itself provides a framework for project people to think about what they have to do, when they have to do it, how much it will cost and what do they need to get the job done. It provides the baseline for scheduling and monitoring the work, and the information to most effectively utilize resources.

Exhibit 12. 1996 Boston to Washington Construction Program

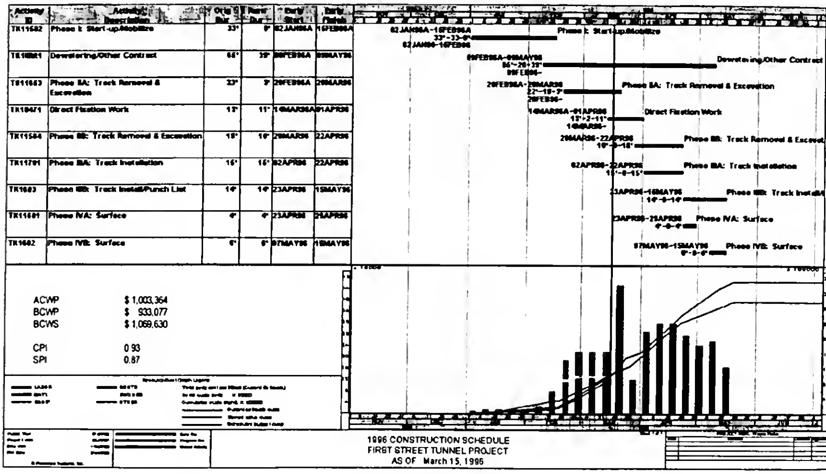


Figure 13. S-Curve with Current Estimate & Earned Value.

The cumulative CPI and SPI are project status snapshots. At a quick glance a manager knows whether the project is moving along as planned or requires closer scrutiny.

In closing, I would like to say that implementation of Program Management with the associate tools and techniques is not a quick and easy solution for a short term fix. It is a long iterative process that requires management commitment at all levels, particularly by executives and upper management personnel.

Most of us in this room have managed a project, and probably feel confident in our management abilities. However, I have found that the level of technical knowledge on project and program management is surprisingly low. Our knowledge of this subject and that of our employees cannot be stagnant. We must be champions for continued education and training. They are key ingredients to implement change and foster improvement to our corporation's bottom line.

Education will also provide a common terminology and criteria. It will allow the organization to speak the same language. Communication in all directions, vertical, horizontal and diagonal speeds up the information flow and decision making. If we want to be competitive in today's world, we must empower the people with the responsibility to seek answers to their questions from the source.

The old practice of "throwing the monkey" over the wall will no longer work. We cannot continue to pass problems along or solve just one element of the problem and let someone else figure out how to make it work. We must organize as a team, working toward a common goal and vision.

Program Management can be the keystone that ties together all the engineering functions, integrating resources into a consolidated effort to bring projects in on time and on budget.

References

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(2) Beyond Communicating with Earned Value—Wayne F. Abba, U.S. Dept. of Defense

(3) Reengineering the Earned Value Process: From Government into the Private Sector—Quentin W. Fleming, Joel M. Koppelman

(4) Using Performance Indices to Evaluate Estimates at Completion: Major David S. Christensen, Ph.d. USAF

CAPACITY IMPROVEMENTS ON SANTE FE'S MAIN LINE

By: W. E. Van Hook*

Good morning ladies and gentlemen. I am honored to have this opportunity to discuss with you this morning the former Sante Fe's main track capacity improvements on its "Transcontinental" main track.

My talk today will focus on the double track construction by the former Sante Fe on the Chicago to Los Angeles route, which is still ongoing with the newly merged Burlington Northern Sante Fe. I will discuss in general terms the process used by the Sante Fe for its selection of where to construct these improvements and touch briefly on how the Burlington Northern Sante Fe now completes its analysis for capacity improvements.

I will close this speech with a brief review of a project completed in 1995 in the State of Oklahoma and entertain any questions you may have at that time.

As I am sure you are all aware, the Sante Fe merged with the Burlington Northern by approval of the Interstate Commerce Commission on September 22, 1995. We now have a combined route mileage in excess of 31,000 miles reaching across twenty-seven states and two Canadian provinces. At this time the two operating companies, the Atchison, Topeka & Sante Fe Railway Company and the Burlington Northern Railroad Company, are separate, wholly owned subsidiaries of the Burlington Northern Sante Fe Corporation. Our combined operating ratio, excluding unusual items, was 79.9% for the year ending 1995.

Figure 1. BNSF Trains

*AVP Construction, Santa Fe Line, BNSF Corp.

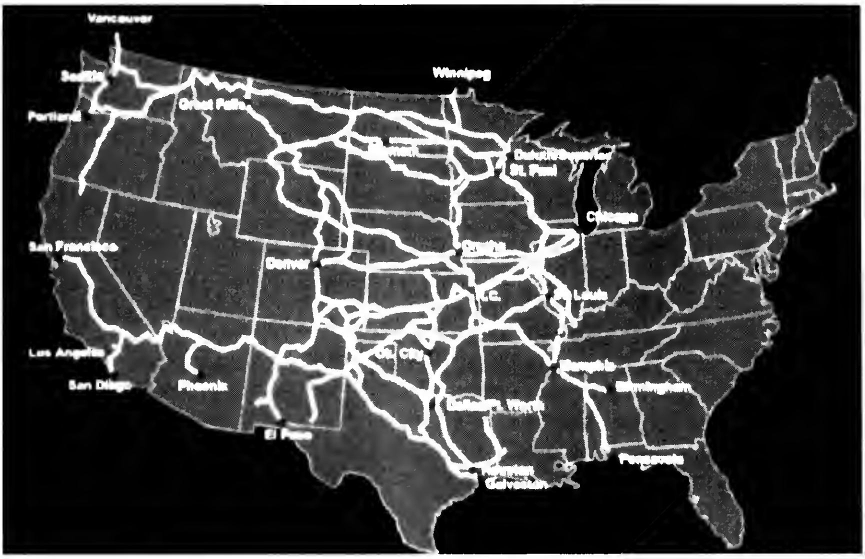


Figure 2. Operating Lines

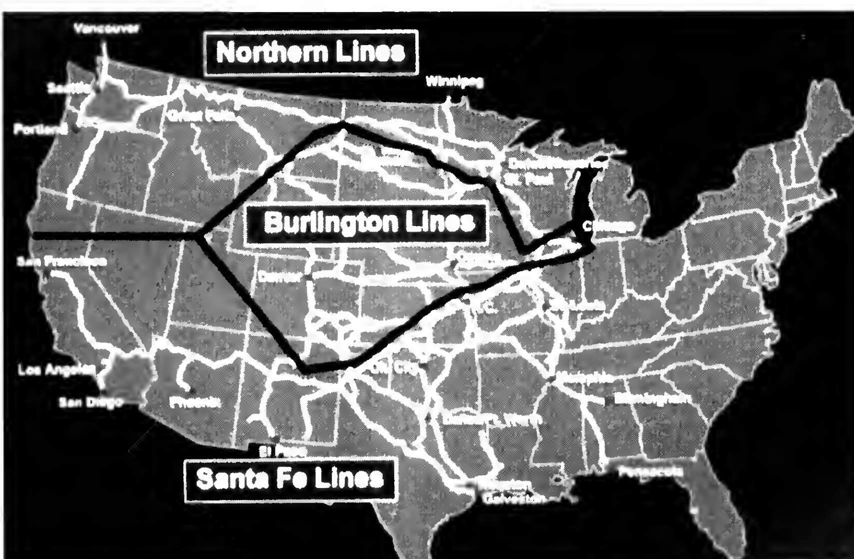


Figure 3. Operating Lines

Expansion Projects ($Millions)	
Northern Lines	$139
Burlington Lines	83
Santa Fe Lines	100
Merger related	48
Yard improvements	
Kansas City	$42
Los Angeles	25
San Bernardino	10
Corwith (Chicago)	8
Barstow	7

Figure 4. Capital Commitments

With the merger, the Construction Team, similar to the Transportation and Maintenance Teams is organized into the three geographic areas shown, namely the Northern, Burlington and Sante Fe Lines.

Our capital budget for 1996 is right at $1-1/2 billion. Of this amount, $462 million has been allocated for Expansion Projects shown here. As you can see, of this amount, $100 million is planned for Sante Fe Lines.

The primary focus route on the Sante Fe was the line from Chicago to Los Angeles, as shown here. On this route, our trains can transit this corridor in forty-five hours, which is the schedule for our Train 981, Los Angeles to Chicago. In any twenty-four hour period, at any particular point, anywhere from thirty to seventy trains will pass, dependent upon the traffic levels for that day. Our Gross Tonnage for this route is between sixty and ninety million on an annual basis. Also shown by the solid lines in this figure are the approximate areas which had double track as of the end of 1995.

Of the approximately 2,200 route miles, we had 494 miles of single main track remaining as of December, 1995. With the 55 miles of double track currently being completed, we will have only 439 miles, or 20%, of the total route remaining to do. You will note that there are several "gaps" in the existing double track locations, primarily in the States of New Mexico, Texas, Oklahoma and Kansas. One could conclude that double tracking was performed at the obvious "choke" points on the property. Those locations being primarily terminals, crew change, interchange and areas of heavy local switching.

Just how is it that these various segments were selected for double tracking by the Sante Fe? How are the locations being selected now by the Burlington Northern Sante Fe? One would think that significant cost savings could be developed by starting at one end and systematically proceeding to the other end. You would also think that significant cost savings could be developed if by nothing else than by reduced mobilization costs from "bouncing" around from one location to the next. What is the driver behind this seemingly random selection of locations to perform capacity improvements?

It is the customer. As shown here, it is "Our vision to realize the tremendous potential of the new Burlington Northern and Sante Fe Railway, by providing transportation services that consistently meet our customers' expectations." Although when the 1995 projects were being developed this statement was not in existence, the Atchison, Topeka & Sante Fe Vision Statement was very similar in so far as why we do what we do - it is driven by THE CUSTOMER.

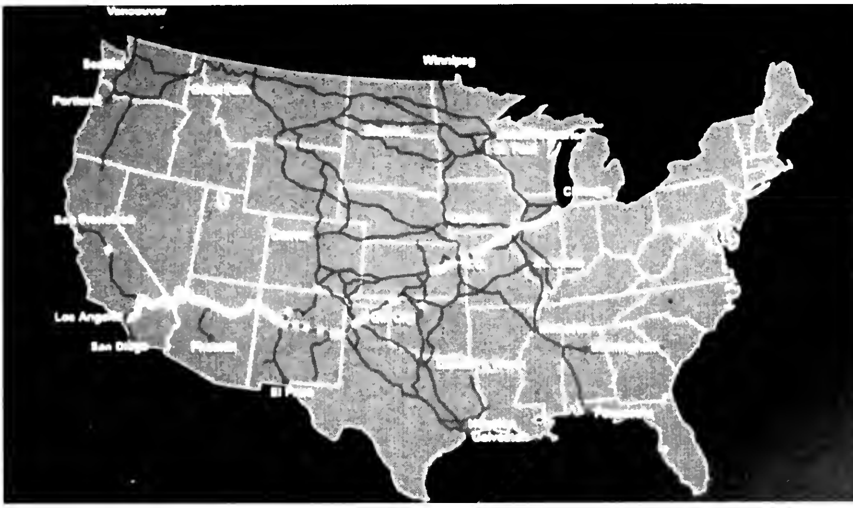


Figure 5. SFL Double Track

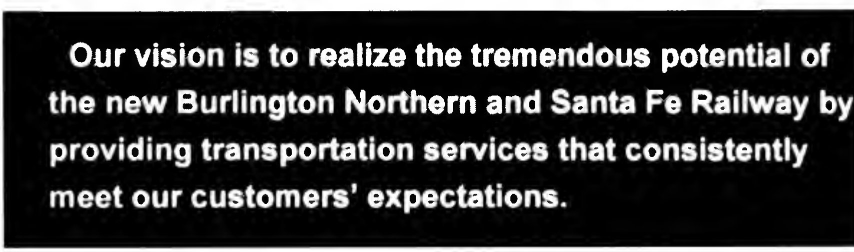


Figure 6. SFL Vision Statement

Figure 7. Customer Expectation

Our planning process for the selection of capacity improvements is geared towards the satisfaction of our Customers' Expectations. Our Business Units are continually involved with the Operating Units in the Strategic Planning process throughout the year. The customers express to us very clearly what their expectations are. We also must factor into the equation our growth projections, new markets and the continual improvement of our service to our customers. Naturally, this includes improved on time delivery, reduced failure rates and reduced damages to any commodity.

From the Strategic Planning sessions, the volumes of traffic and scheduling improvements are conveyed to our Strategic Analysis Modeling team. Our Strategic Analysis Modeling team is headed up by an Asst Vice-President who reports to the Finance Department. The Modeling team then simulates, in priority order, the various scenarios developed by the Resource and Business Units. Once the necessary capacity improvement areas are determined, cost estimates are developed, and financial analysis is completed. This information is then passed back to the Business and Resource Units for incorporation into their Strategic Planning. Those with the highest return on investment and/or savings to our operating expenses are then recommended by the Resource Units to the Capital Budget Team for development of the annual capital budget report for submission to Senior Management and the Board of Directors.

But how is the modeling performed to meet the requirements of the Business Units?

As shown here, there are two models currently used by Burlington Northern Sante Fe in our analysis. From the former Sante Fe comes the Dispatch Planning Model. Whereas, from the former Burlington Northern, the Line Capacity Model was used. With the merger, we now have the ability to utilize either model, dependent upon the specific characteristics of the situation being considered.

For both models, physical plant data is input along with the timetable operating constraints, slow orders and General Orders recently issued. Extensive interviews are also conducted by the Modeling Team with the dispatchers, planning and field transportation teams to review, and solicit from them, problem areas or concerns for implementation into the modeling process.

Both models are also fed data from the Train Performance Model which includes train characteristics for the line segment being analyzed.

For the train characteristics input, detailed data such as the types of locomotives, horsepower per ton, type of car, commodity, and so forth are input for the particular line segment being studied.

For these two models, lets look at the Dispatch Planning Model first.

The Dispatch Planning Model is a schedule based model. By schedule based, we mean you have to tell the model what each train entered will do as it passes through the line segment being considered. The train movements are input with scheduled running times from point to point and delays predetermined, such as crew changes and addition of helper locomotives.

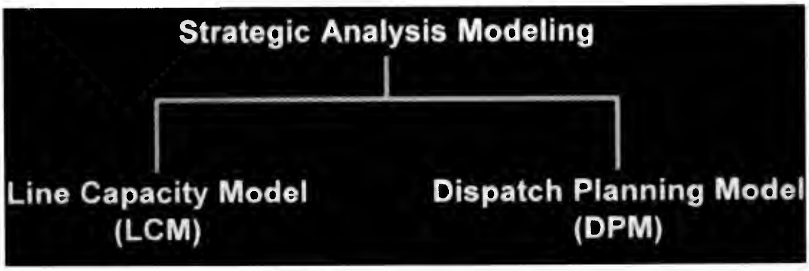


Figure 8. Strategic Analysis Modeling

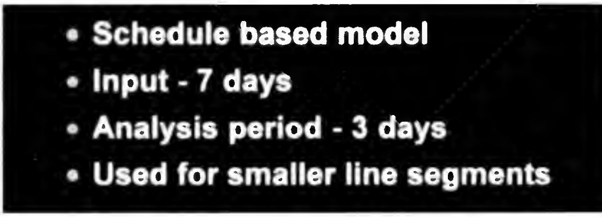


Figure 9. Dispatch Planning Model

The input data may be for a long period of time, but the analysis of this simulation is limited to seven (7) days. During the analysis, the model reviews any continuous three day period desired with the analysis being focused on the second, or middle, day.

This program was used on the former Sante Fe with the Chicago to Los Angeles corridor being the primary focus route, as stated earlier.

For the proposed capacity improvements, the output will provide you with the train hour delays for each particular point-to-point segment that is being considered. Similar to the use of the cursor on a slide rule, for those who have used one, as you move the capacity improvement segment along the route, you will be able to select the location which generates the largest reduction in train hour delays.

How does this compare to the Line Capacity Model?

The Line Capacity Model is the model that was developed on the former Burlington Northern Railroad. This model also accepts scheduled train input, but has the advantage of also including in its analysis via the random events generator, random events that model real life train movements.

The random events generator will make a delay occur to train moves based upon the input entered for the system. For the former Burlington Northern, in 1981 failure data was collected and then again verified in 1994. Based upon these studies, between 1% and 5% of all trains will incur a delay at any particular time.

As an example, the types of random events applied would be signal failures, broken rails, hot box detector false readouts, broken drawbars, and locomotive and car defects, to name a few.

The Line Capacity Model will also apply random trains such as locals, work trains and unscheduled unit train moves.

With the ability to include random failures and random train moves, the model can predict what is required to compensate for that level of failures and random trains specified, which closely simulates actual operations. For these reasons, coupled with the model's ability to analyze a thirty day window of analysis and its graphics ability, this model will be used primarily for analysis of the longer and more complex route segments for the Burlington Northern Sante Fe.

The Dispatch Planning Model will continue to be used for the shorter line segments where the main consideration is the impact upon scheduled trains.

For 1995, the Dispatch Planning model showed that the location with the largest train hour delays was occurring between Heman and Curtis, Oklahoma.

The limits of this project were from the east switch of Heman to the west switch of Curtis, located in the State of Oklahoma on the Panhandle Subdivision located about mid-point between Amarillo, Texas and Tulsa, Oklahoma. The Heman to Curtis capacity improvement project was a total of 16½ miles long, with 8.5 miles of existing siding upgrades and 8.0 miles of new embankment

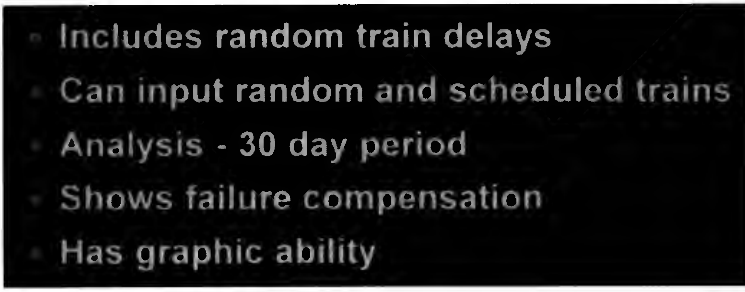


Figure 10. Line Capacity Model

Figure 11. Heman/Curtis

construction. This work cost us $19 million, which included all facets of the construction, including bridge and signal upgrades as well as the installation of universal No. 24 crossovers.

The Heman to Curtis area had the highest level of train delays, therefore becoming the highest priority for double tracking for our traffic base as of October, 1994. Secondary, but also critical areas, were also recommended. Pedernal to Carnero, New Mexico and Pampa to Hoover, Texas, for an additional 23½ miles of doubletrack. In late October, we released our Engineering Consultant, Hanson-Wilson, to begin design on the project. To meet the overall project goal of in-service before the Christmas rush of October 1, 1995, the project was put on an expedited schedule.

The engineering was completed in the first week of February, 1995 and was immediately put out for bids. Bidding was completed in early March, 1995 and a notice to proceed was given to the successful bidder, Neosho Construction, on March 15, 1995. In order to provide enough time for our track and signal forces to complete their work by November, the contractor was given until August 1, 1995 for substantial completion of the work.

The team that was responsible for this project being completed on time consisted of the following:

- Company Employees—Provided overall project supervision and management, along with all track, signal and surfacing gangs for construction of the project from the sub-ballast up.
- Hanson-Wilson, Inc.—Design and Construction Management Services. Hanson- Wilson was retained to provide all design services, permitting and construction management of grading, bridge construction, pole line removal, project closeout and all surveys.
- Neosho Construction—General Contractor. Neosho provided construction of all new embankment, structures, grade separations and asphalt subballast.

As in all projects, you can control your own forces, your engineering consultants, and most of the time, your contractors. The only thing you can not control is Mother Nature. Normally, for this area of Oklahoma between April to August, it rains 29 days, for an average of 17.62 inches. In 1995 it rained 50 days for a total of 28.39 inches. Even with Mother Nature trying her hardest to cause delay, with plenty of extra effort by the entire project team, this project was completed on time and within budget.

Figure 12. 2nd Heman/Curtis

Figure 13. Grading

Figure 14. Grading Landscape

The hard work of this team put together a high quality project. In total, the team moved 512,350 cubic yards of cut, and placed 298,000 cubic yards of fill, to construct new embankment or widen existing roadbeds. The 16 mile project was built on 15 foot track centers with 2 to 1 fill side slopes and 1 to 1.5 cut side slopes.

In total, 38 bridge structures were needed in this project. These structures varied highly in size and type, but fall into four general areas. The first major bridge type was grade separations. Two replacement highway overpass grade separations were built, since the existing grade separation substructures were too narrow to allow for the second mainline.

Lengthening the existing structures was ruled out due to their poor condition. The two replacement structures were paid for entirely by the Sante Fe in order to meet the schedule.

The new structures consisted of cast in place concrete piers and abutments, on steel bearing piles.

The superstructures were precast prestressed I section with cast in place concrete decks. Another major bridge type was concrete box beams. A total of five of these structures were constructed.

The other three grade separations were major drainage structures.

This structure type consisted of driven H-section bearing piles, with precast concrete pile caps and abutments.

The superstructures were precast prestressed concrete double voided box beams, with integral walkways and curbs.

Our next major bridge type was cast in place concrete box culvert extensions. Three new structures were cast to match existing box culvert openings.

Our last structure type was miscellaneous drainage culverts. A total of 28 were placed. Existing culverts were either extended in kind, or replaced with new bored steel casement pipes under the existing embankment, and thru the new embankment as well. Boring of new culverts was used if the existing structure was in poor condition, or needed to be adjusted to match present waterway flow lines.

Figure 15. Overpass Cuts

Figure 16. Bridge Before

Figure 17. Bridge After

Figure 18. Bridge After

Figure 19. Bridge Before

Figure 20. Bridge After

Figure 21. Concrete Box

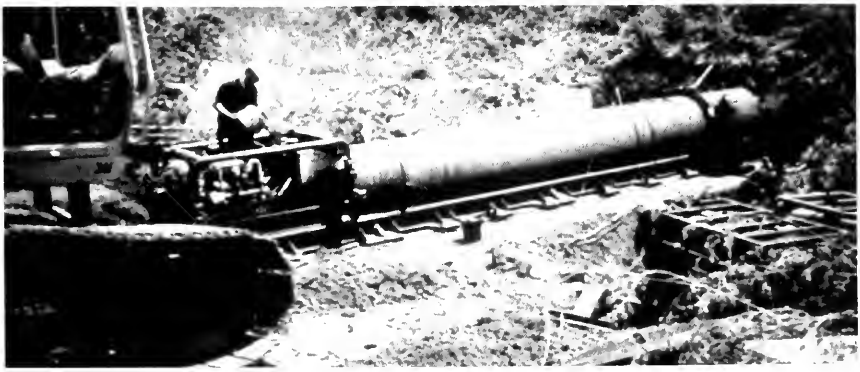


Figure 22. Pipe Bore

Figure 23. Grading

One of the more unique aspects of this project was the sub-ballast. The soils is this part of the country are silty sands with highly plastic clays. The strength of the soils did meet design loading at optimum moisture, but the strength fell off greatly above optimum moisture. Two options were presented to us for subgrade modifications. The first option consisted of a minimum of 12 inches of rock sub-ballast. The second option was installation of 6 inches of hot mix asphalt subgrade.

The overall economic analysis determined the asphalt option was best suited for this project. One benefit that was not realized until well into construction, was that no matter how much rain we received the track laying machine could still operate on the asphalt surface.

On several occasions, due to muddy and impassable assess right of way roads, our Company forces could not get to the project site by truck and had to walk in on the asphalt to start work. If we had not placed the asphalt we would have lost several days of production. The only down side of the asphalt is that it requires tighter tolerances of subgrade finishing, than needed for conventional track grading.

Figure 24. Asphalt Truck

Figure 25. Long Asphalt Road

The finished asphalt surface was built to meet a quarter inch deviation from design grade in a 10 foot section. This insures that the concrete ties will sit squarely and uniformly on the surface, thereby not sustaining any damages during track laying operations, and allow for proper seating of the rail base onto the ties.

New trackage was constructed with concrete ties manufactured by Rockla using Pandrol fasteners and twelve inches of ballast under the concrete ties. We leased a Fairmont (TLM) track laying machine, with tie cars, of sufficient quantity to have ten cars with the TLM every day to place the ties.

With the ten cars a day, 2,200 ties could be distributed on 24-inch centers for approximately three-quarters of a mile of track a day. 136-pound welded rail was installed with the TLM. The upgrading of existing sidings consisted of installation of wood ties as required, with new 136 lb rail used to relay the entire siding lengths. Sidings were surfaced to match new design grades as well as to improve drainage.

Figure 26. Asphalt w/ train

Figure 27. TLM

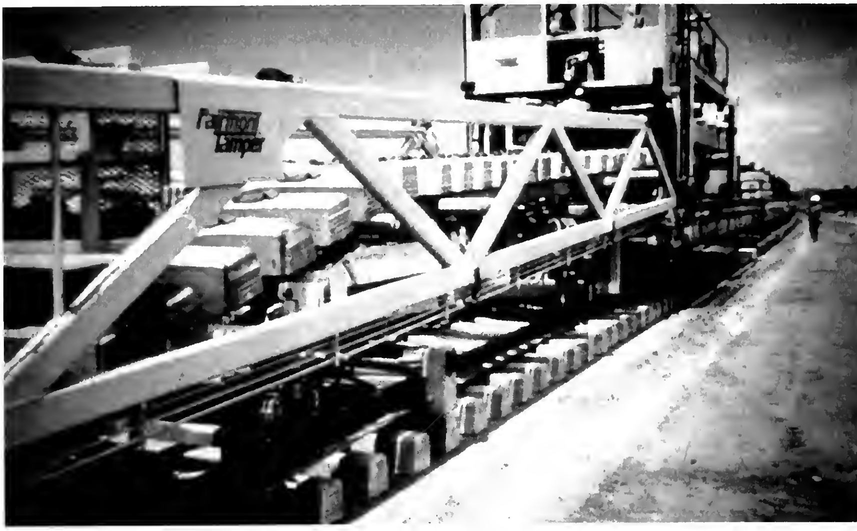


Figure 28. TLM

Figure 29. TLM

All turnouts for sidings and setout tracks were upgraded to No. 14, and No. 24 turnouts were placed at the ends of double track sections. Double universal No. 24 crossovers were also installed at Belva, Oklahoma. The entire project signal system was upgraded to a data radio controlled C.T.C. system.

We did complete this project on schedule, however, with significant overtime due to the heavy rains. This project was also completed within budget. This project also assisted the Operating Team in meeting our goal of handling 27,040 trailers without any service failures for UPS during its Christmas season traffic.

Figure 30. BNSF

For 1997, we are currently collecting preliminary engineering information to install another 55 miles of second main track. The only change on our design parameters will be the change to twenty-five foot track centers wherever economically justified. We will be attempting to have our track centers transition in existing curves wherever we have constraints not allowing us to construct trackage at twenty-five foot centers. For our roadbed width, we will be providing for fourteen feet from centerline of new track to top of slope of new roadbed.

The locations being considered are two in New Mexico and one each in Texas and Oklahoma. Naturally, these locations could change significantly once the analysis and strategic planning process is progressed within the next two months.

In closing, I hope that I have conveyed to you the capacity improvement process as used on the former Sante Fe and now used on the Burlington Northern Sante Fe. I feel that these are exciting times for all of us in the railroad transportation industry. It was not that many years ago that we were talking about and executing downsizing of our physical plant. As we continue to progress these various expansion projects, I believe that we must all remember that we must deliver a cost efficient, quality product in a safe environment, without interference to our Transportation Team, to our Customers—Our customers being our shippers, our stockholders and all of the other departments of our respective Companies. Lest there be any doubt in your mind, rest assured that if we don't - someone else will.

RECONSTRUCTION OF THE CTA GREEN LINE THE SECOND CENTURY

By: George C. Haenisch*

The Chicago Transit Authority operates over 240 revenue miles of rapid transit trackage. CTA provides rail transit service within the City of Chicago and seven suburban communities. Service, for the most part, is operated 24 hours per day, every day of the year. There are seven separate lines, designated Red (North-South), Orange (Southwest), Yellow (Skokie), Green (West-South), Blue (Northwest-West-Southwest), Purple (Evanston-Wilmette) and Brown (North-Northwest). The revenue trackage is evenly divided between open deck elevated, subway and ballasted.

The majority of CTA's trackage, and all of the open deck elevated, predates the creation of the CTA in 1945 (Referendum 6-4-45). Actual operations began on 10-1-47. The Green Line route, which comprises approximately one seventh of the CTA Rapid Transit Right-of-Way, includes the oldest open deck elevated structure in CTA's system. The original South Mainline structure was constructed by developers in conjunction with the southward growth of the city and the Columbian Exposition world's fair in the early 1890's. The Lake Street line was constructed by another predecessor to the CTA in the early 1900's and normal track renewals, there have been no major renovations to the Green Line since World War II.

The last major tie renewal on the Green Line took place in the 1960's. At that time, the tie renewal utilized creosote pine ties, cut track spikes and clamp on rail anchors with an anticipated service life of 25 to 30 years. Between 1989 and 1992, CTA Engineering commissioned outside consultants to prepare Engineering Condition Assessment (ECA) reports on the entire CTA Rail System, to determine the current physical condition and anticipated remaining life of the rail infrastructure. This assessment was conducted on a bent-by-bent basis, and track, structure, systems and facilities were evaluated and rated as being in good, fair, moderate, poor or critical condition.

As a result of the ECA report and further analysis and prioritization of needs by CTA and its Construction Program Management consultant (CPM), it was readily apparent that the Green Line was in critical need of major rehabilitation. The Green Line contained over 50% of the entire rapid transit system slow zones. The condition of the 30+ year old ties, 40+ year old rails, 50+ year old special trackwork and a structure between 90 and 100 years old resulted in one third of the Green Line being under slow orders. CTA and CPM established early, as an objective, to perform the necessary rehabilitation to return the Green Line to forty more years of useful life.

At the same time as these evaluations were being conducted, CTA recognized that infrastructure improvements alone would not provide patrons with a tangible improvement in transit service. Along with upgrading the right-of-way, the transit stations themselves, many of which also dated back to the construction of the line, were in need of improvement. Stations were inadequately illuminated, modern public accommodations were lacking, and many of the stations were not compliant with the Americans with Disabilities Act (ADA).

In the fall of 1993, CTA made the decision to embark upon a 2 year, $300 Million rehabilitation of the Green Line. Based upon studies by the CPM, the Authority deemed it appropriate to shut down transit service during the rehabilitation, to reduce the anticipated cost of the work by 50% and to reduce the overall period of disruption to the ridership and the communities where the construction would take place. CTA and CPM organized teams to fine tune the scope of work and to put the project on a fast track. While final engineering work was being completed, the line was shut down and demolition work began.

*Vice President for Construction and Maintenance, CTA

In early 1994, CTA and construction contractors began the process of removing trackwork, salvaging system components and securing stations in anticipation of the rehabilitation work. In an innovative approach, CTA forces and contractors worked cooperatively to execute the work. CTA forces removed trackwork on the Lake Street branch, secured transit stations, salvaged signal and traction power equipment while contractor forces performed heavy demolition work and removed trackwork on the South Mainline.

While demolition work proceeded, two engineering teams engaged by CTA (Rust E&I and Chicago Transit Partners) prepared final construction documents for the rehabilitation of structures, replacement of trackwork, traction power systems and signal systems. CTA and the CPM used engineering documents prepared prior to the conception of the Green Line project to "fast track" critical protective coating work, column base renewal and long lead track materials procurement. By the spring of 1994, demolition work was substantially complete and three protective coating contractors began the process of removing 100 years of accumulated dirt, rust and paint, much of which was contaminated with lead.

The protective coating work required the contractors to employ innovative techniques in enclosing the structure to contain the hazardous lead paint dust, while still permitting traffic on the city streets below the structure. Each of the protective coating contractors used unique methods which best suited their part of the structure (which were constructed to fit different site conditions). CTA forces also conducted a well choreographed replacement of track structure elements in conjunction with the protective coating work. In the summer of 1994, the second wave of contractors came on line. These contractors were responsible for the major structural rehabilitation work, including the replacement of two truss structures with through plate girders on the Englewood branch, and the total reconstruction of spans at a curve near 40th and Michigan on the south side and at Laramie on the west side (where the right-of-way transitions from open deck to ballast). Two other contracts were responsible for the reconstruction of column bases and foundations.

By the end of 1994, the structure rehabilitation work was well in hand, protective coating work had been completed on the top structure surfaces and running rail, special work, track ties and contact rail were either on hand or in fabrication in anticipation of the reinstallation of trackwork in 1995.

In addition to the replacement or rehabilitation of track and structure, major improvements were planned to the traction power and signal systems. On the South Side, four new traction power substations were constructed, major elements of the signal systems at the 36th/37th Street middle track, 59th Junction and Ashland Yard were upgraded, signal cable and traction power distribution cable was replaced, and equipment upgrades were identified for three other substations.

On Lake Street, the entire cab signal system was replaced, improvements made at substations, signal cable and traction power distribution cable was replaced, and the shops at Harlem Yard rehabilitated. Detailed system design and fabrication of long lead components were begun in late 1994 and early 1995 so that these systems would be installed immediately following trackwork.

After evaluating alternatives to creosote pine ties used in the last tie replacement, CTA selected azobe ties in critical areas over city streets and douglas fir ties in other areas. In late winter 1995, the two track contractors began the installation of new track and rail. Due to the difference in structure components, the contractor on South Mainline was able to employ a panelized method for the track installation. On Lake Street, where clearance to adjacent structures was tight, the contractor hand laid ties.

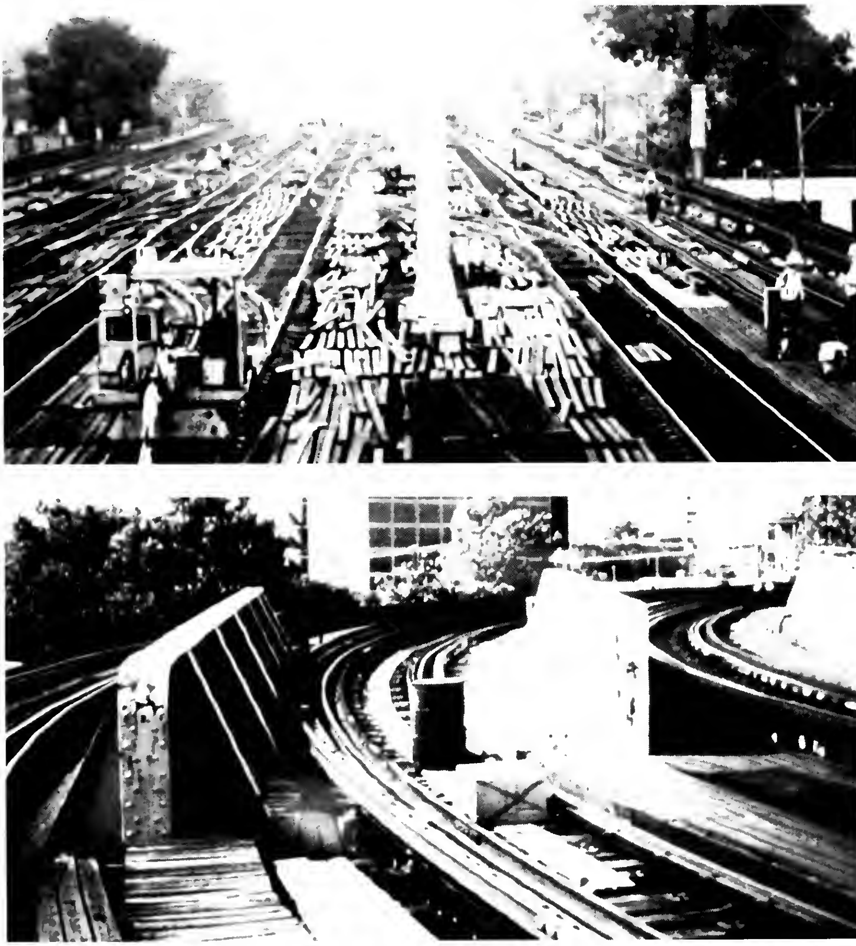
As weather improved, the traction power contractor began the construction of the new substations, while power conversion and rectification equipment was being factory fabricated and tested.

By late spring, track and rail was well underway. At this time, the first of the station construction contracts was awarded. CTA's original concept for the Green Line was to locate stations on a one mile grid. Because of the impact that transit stations have on the local communities, CTA engaged the local neighborhoods over several months to determine the best location and scope for the stations.

Reconstruction of the Green Line proceeds at various locations.

The result of this effort was an increase in the total number of stations on the Green Line (outside the Loop stations common to several other lines) from 18 to 26. CTA was able to identify funding for the rehabilitation or construction of 22 of these stations.

Two of the stations to be constructed, at Pulaski Avenue and Garfield Boulevard, would be transit centers which would incorporate space for community-focused amenities in addition to transit service. The line terminus at Harlem-Marion will be the site of a multi-modal station, serving CTA rail and bus, commuter rail (METRA) and suburban bus (PACE). New transit stations were designed for Morgan, California, Kedzie, Laramie and Cicero on Lake Street, Halsted on the Englewood branch



Reconstruction of the Green Line proceeds at various locations.

and 51st Street on the South Mainline. The existing stations at Central, Ashland/Lake, Clinton, 35th Street, 43rd Street, and 47th Street were in need of major reconstruction. Stations at Oak Park, Ridgeland, Austin, Indiana, King Drive, Cottage Grove and Ashland/63rd were slated for rehabilitation.

By late 1995, infrastructure work was completed and testing begun on traction power and signal systems. In addition to the transit station upgrades, CTA installed a new fiber optic-based communications backbone to connect the transit stations with the new Control Center constructed under a separate project. CTA also is modernizing its fare collection system. When the Green Line stations are completed in 1996, they will incorporate state of the art audiovisual signage, connected by computer to the Control Center. Operators will be able to selectively broadcast audio and text messages to patrons. Train arrivals will be automatically announced. All stations will incorporate new graphic signage and tactile edges on platforms which are fully compliant with ADA (Americans with Disabilities Act).

BURLINGTON NORTHERN SANTA FE GPS SURVEY PROJECT

By: Stacy J. Sauer*

Abstract

In 1988 the former Burlington Northern Railroad (BNR) began a GPS (Global Positioning System) surveying project to collect the data needed for the Advanced Railroad Electronics System (ARES) and the Rail Garrison project. BNR has used differential GPS surveying to map track geometry and track features in every year since 1988, except 1993. GPS surveying in 1994 and 1995 will provide data for the Positive Train Separation (PTS) project and other corporate needs.

Introduction

This paper will describe the history of the project and share some of the experiences of the data collection effort.

Railroad engineering departments historically have had a record keeping responsibility for most of the physical assets of the company. Primarily this is due to the fact that railroad engineering departments also maintain and manage the construction of the non-moving physical assets. And, railroad engineering departments have been responsible for reporting the track miles for annual company reports.

One of the most useful documents for company wide use is the condensed profile or track chart containing a simplified plan view of the track, plus grades, curves, stations, mileposts, rail consist, and maintenance history. How do we keep this very useful document up to date?

Originally, records were kept on maps and in ledgers. With the advent of computers, the ledgers went to computer data bases and the maps are gradually being converted to CADD. However due to downsizing and mergers, the field staff available to gather and report the physical changes back to those responsible for maintaining the maps and records has all but disappeared.

Engineering departments must continue to be creative in finding ways to capture changes occurring in the field. New technology on track geometry cars and the new track strength analysis cars allow us to see micro changes in track geometry, track strength, and component wear.

But, what about when the Division moves a cross-over, decides to extend a siding, or removes some tracks without an AFE? We all know this is not supposed to happen but it still does.

In order for engineering departments to keep pace with the field changes, we need tools to track the physical changes occurring due to either capital or maintenance projects.

A small number of railroads have recognized this and have their own dedicated survey vehicles or contracted for mobile mapping services. One railroad has combined on a single hi-rail vehicle the clearance and track center measuring with a GPS survey capability.

Much has occurred in the last several years in the area of mobile mapping technology. State DOT's, counties, cities, and various highway authorities have been building digital representations of the transportation networks they manage. GIS's (Geographical Information Systems) have given these agencies the ability to make better decisions about how to manage their resources. But a GIS needs spatial data as a base, and many of the records of these agencies were built from different datums just like the railroads. GPS allows everything to be referenced to the same datum.

Mobile mapping vehicles have proliferated because of the demand to collect data to build Geographical Information Systems. One company has now built over 50 vehicles with GPS, video, and other data collection sensors.

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Acknowledgments

Some of the people and companies that have played a key part in this project are listed below. They can be contacted for more details than I am able to provide.

Burlington Northern Santa Fe Railroad
Ft. Worth, Texas
These persons are actively involved with the Positive Train Separation Project (PTS).
Nick Marsh, General Director Engineering Technology
Tom N. Atkins, Assistant Director Train Control Systems
Larry Milhon, Assistant Director Network Control Systems

Ohio State University Center for Mapping
Columbus, Ohio
614-292-1600
The Center for Mapping pioneered the systems that are on the currently contracted vehicle and the software used to process the collected data.
Dr. John Bossler, Director

General Railway Signal Corp. (GRS)
Rochester, New York
716-783-2000
Prime contractor for 1994 and 1995 survey. Owner of GPS vehicle and systems integrator.
Mike Dwyer, Acting Vice President New Ventures
John Schultz, Program Manager
Fouad Khalil, Technical Manager

TransMap
Columbus, Ohio
614-487-3636
Subcontractor for GPS data processing and technical support.
Kurt Novak, President
Gabor Blaho, Chief Systems Engineer

P&L Systems, Ltd.
Columbus, Ohio
614-891-4970
Subcontractor for GPS field services.
Thomas E. Rybski, President

The following companies are not currently on the project but must be recognized for their past contributions:

GPS Technology Corporation
Houston, Texas
713-589-6700
Former GPS contractor to Burlington Northern from 1988 through 1992.
Bill B. Crow, Chairman

GPS Overview

This paper will not attempt to cover explanations of the GPS system or GPS surveying techniques. There are many good sources of information and the Internet has several good sites. I would recommend the following sites for information regarding GPS:

GPS—General Information Sites:
http://www.inmet.com/~pwt/gps_gen.htm

GPS—University of Texas, Austin:
http://www.utexas.edu/depts/grg/gcraft/notes/gps/gps.html

History of GPS Survey Project

I have divided the history of the survey into two parts. Part one is from the beginning in 1988 through 1992. Part two are the years of 1994 and 1995. The first period of surveying took place during the time when the NAVSTAR constellation was still being built, mobile mapping technologies utilizing GPS were still in their infancy, and 1st order control points needed for differential GPS were sparse. By the end of 1993, a number of things changes and improved. The NAVSTAR constellation was complete, many companies had gained experience with mobile mapping systems, and 1st order control points set or readjusted using GPS were becoming plentiful in many parts of the country. After 1993, it was time for BNR to reassess how to continue the GPS surveying project.

GPS Survey—1988 through 1992

BNR's GPS surveying project through 1992 consisted of using Chevrolet Suburban equipped to "Hi-Rail" the track, (in some years, 2 vehicles were used) 2 Sercel GPS receivers with antennas, 2 laptop computers for monitoring the satellites, and a laptop computer to log events. The contractor (GPS Technology Corp. of Houston, Texas) used 3 base stations running simultaneously within 250 miles of the vehicle for differential mode surveying. Specifications were for 1 meter horizontal accuracy. This survey typically completed 400 to 500 miles per month, working a continuous 20 days out of the month. Productivity depended on satellite window time availability, number of features to stop at, and track time availability. The best satellite windows were occurring at night during the months that the survey was taking place. Our survey crews were frequently working during the early A.M. hours.

The vehicle would travel from track feature to track feature at 25 MPH. All features required a stop of 2 minutes. This time was used to add attribute information about the feature to the laptop computer software. Loss of lock with the satellites was typical at overhead bridge structures and near some buildings requiring another 2 minute stop to regain lock.

All passing sidings and second main tracks were also surveyed, requiring the vehicle to make a second pass.

Route mileage surveyed during these years is as follows:

1988	200 miles	1991	2000 miles
1989	415 miles	1992	4000 miles
1990	640 miles		

GPS Survey—1994–1995

With the increases in our train traffic levels the past few years, it was going to be very difficult to continue to survey in the previously described manner. We needed a way to survey that caused little interference to the train traffic. We needed a way to obtain the feature attribute information after we got off the track.

An RFP was let in 1994 for GPS surveying on BNR. With the recent advances in mobile mapping systems, we felt there were companies that could meet our objectives. General Railway Signal (GRS) was selected from seven bidders.

The current GPS surveying project supplies the following:

Video, 4 directions.

Digital Stereo Images used for obtaining 3 dimensional coordinates of features.

A 3 dimensional coordinate at least every 50 feet along the center of the track.

A "down-track" distance between every 3-D coordinate and feature.

Other derived fields like UTM coordinates, instantaneous grade, azimuth, and curvature.

Testing of the new mobile mapping system began in August of 1994 with implementation on BNR in October. An initial 700 mile route was selected in the Midwest. This route would tie together several routes that were surveyed in previous years by a different contractor.

Comparison of the GPS coordinates produced by the new mobile mapping system and the previous 1992 GPS survey at overlapping points produced good results. Comparison of the center-lines of the track between the two surveys ranged from 2 inches to 40 inches.

We felt there was good enough correlation to continue the survey. The vehicle and crew was moved to the state of Washington to begin surveying the trackage where the PTS (Positive Train Separation) tests would be performed.

In December of 1994, the survey began at Seattle. Only 230 miles were surveyed. Many problems were incurred. Traffic levels were at an all time high on this portion of our railroad. The weather was not cooperating as could be expected in western Washington. Obstructions to receiving enough satellites were encountered in many places. And we were experiencing some system problems on the vehicle.

During the down time between the 1994 and 1995 surveys many improvements were made to the mobile mapping system. Reliability issues were addressed, improvements were made to the analog video cameras, and system start-up and shut-down steps were drastically reduced.

Route mileage surveyed during these years is as follows:

1994 1000 miles

1995 7150 miles

Current Mobile Mapping System Components

The vehicle is equipped with 4 video cameras. The cameras point to the front, back, left, and right sides of the vehicle. The video tape format is Hi-8. There are two digital cameras pointed towards the front to record the stereo images (Figure 1).

Five VCR decks are used to record the analog video. The fifth recorder is to record an extra front video as a backup. The front video has route information encoded on it.

Figure 1

The stereo images are recorded on a 5 gigabyte DAT tape. The tape drive has two drives with the second drive used for mirroring the data to a second tape.

Gyroscopes and a wheel counter system allow the vehicle to travel up to 2 miles without GPS lock.

Because the NAVSTAR constellation is now full, we can survey at almost anytime of the day. But the addition of analog video cameras and digital cameras restricts the survey to the daylight hours. The weather is now a factor also. Raindrops do not degrade the video image appreciably, but they adversely affect the digital stereo images.

The direction of the sun also plays a factor in planning the survey route. Like the rain, the sun affects the digital cameras more severely.

Communications, Base Stations, and Control Points

Many of the mobile mapping projects that I am aware of do not use differential surveying techniques. Because of the PTS requirement to determine which track a train is on, we have to use differential. This adds a lot more to the cost and complexity of the field operations. A lot of planning is required and coordination with all those involved.

The analog video cameras and the digital cameras allow for continuous surveying without stopping. The current contractor's requirements are for the two base stations to never be more than 50 miles from the vehicle. Communications between the vehicle and the base stations has been a problem from the very start. Pagers and cellular phones are only reliable within metropolitan areas and much of the surveying is occurring in rural areas. The best solution has been to set up a voice mailbox on BNR's voice mail system. The survey vehicle updates the voice mailbox on it's progress. The base stations use their cellular phones or a nearby public phone to call in to the voice mail to find out about the vehicle's progress. Cellular phone use is somewhat expensive so we have now installed 30 watt Motorola radios in all the contractor's vehicles. BNR has a mobile radio telephone system called MRAS. The radios are tried first. If communication with the radios is not successful, we fall back to the voice mail plan.

All the base stations and the vehicle use the Trimble 4000 SSE type GPS receiver.

NGS (National Geodetic Society) HARN (High Accuracy Reference Network) control points are used whenever possible. In some states like Washington and Oregon, these 1:10,000,000 order control points are well established. Other states like South Dakota require setting some of our own control points.

Quality Control

As an independent check to the coordinate points generated by the vehicle, quality control check points are set approximately every 20 to 40 miles along the survey route. These are temporary control points that can be seen by the vehicle's digital cameras. They are set using one of the GPS receivers used for the base stations. While post processing the data, the contractor uses the stereo image software to determine the coordinates of the quality control points. These coordinate values are compared to the post-processed GPS data. The difference averages around 20 cm.

At every opportunity, the GPS survey will try to overlap a previous GPS survey. At some junctions we do not always get the opportunity due to track availability. As stated previously, we have achieved good repeatability, generally from 2 to 40 inches.

Field Operations and Experiences

There are some major differences in operating a mobile mapping system on a railroad compared to roadways. Vehicle selection is more limited. The van type vehicles can not be fitted with the additional wheels needed for operating on the rail. You can't just pull up to the track, get on and take off. Proper authority is required from the dispatcher. And once you have that authority, you must get on the track immediately and be ready to survey, or else risk losing your "window of opportunity". Many times due to the amount of train traffic, there will only be certain windows available for surveying. The windows are not always reliable due to many factors. Dispatchers do not always want to cooperate. They have their own goals to meet and those goals are to keep the trains running on schedule.

The logistics for operations can be overwhelming at times. All of the following must be considered: track time availability, dispatcher notification, yardmaster/trainmaster notification, scheduling division personnel for pilots, finding and occupying base station control points, avoiding low sun angles, rain, vehicle and equipment maintenance.

Information on the location of control points sometimes is not reliable. Not all control points are suitable for GPS because of obstructions or their order of accuracy. An occasional landowner may have destroyed the control point. Control points were not always ideally spaced or accessible along the track.

Track inspectors or section foremen are provided as pilots. When approaching a terminal, we sometimes have to line up 3 pilots in case we make good progress. Unfortunately, sometimes we are delayed and can end up causing Division personnel to wait for us.

Even though the mainline dispatcher may be helping us make progress, he has no control over the yards. This is where we frequently have the most delays.

There were locations where we started out early with the sun behind us, but due to train delays, by the time we were able to obtain track authority, the sun was now pointing toward the front cameras.

Usually the division personnel (roadmasters and their pilots) were very helpful in getting us across their territory. Due to an occasional system malfunction on the vehicle, we would need to resurvey. Suddenly, we were just another roadmaster's headache.

USES FOR THE DATA COLLECTED

Train Positioning

During the first period of surveying (1988–1992) the primary use for the GPS survey data was the Advanced Railroad Electronics System (ARES). ARES was intended to provide train location and train speed to a command and control system via digital communications using BNR's existing microwave and fiber-optic network. A traffic planner would produce the optimum plan for operating trains over the whole system, slowing some trains, speeding up or rerouting others, scheduling repair crews onto the tracks and then getting them off in time for the next train to pass. ARES was tested but never put into production because of budgetary reasons.

More recently, the Positive Train Separation (PTS) project will be used to test systems to prevent train collisions. The GPS data collected will provide a base map to compare the actual position of the train to surrounding features, like signals, switches, and track restrictions. The GPS data required for on-board positioning and enforcement will be maintained in a central office and downloaded selectively to locomotives at their points of origin. Positioning data sent from equipped trains to the central office will allow the central office to have a graphical display of the trains position, calculation of estimated times of arrival, and assist network planning.

Training

Since BNSF's dispatchers are consolidated into two offices, they will not have as many opportunities to actually see the territories they dispatch. The analog video will be used to familiarize the dispatchers with their territories.

Asset Management, Line Improvement Studies

BNSF's engineering department maintains the track, bridges, buildings, signals, and telecommunications equipment for the railroad. In the late 1980's, the engineering department began to consolidate the data for all the physical features of the railroad into the Roadway Information System (RIS) database.

Data is collected from the GPS survey for the following features:

Bridges
Clearance (plus or minus 6 inches)
Electronic detection or warning devices
Diamond crossings (railroad crossing another railroad)
Mileposts
Road Crossings
Signals
Switches
Signs

By reviewing the video and the track chart, a person can quickly determine where the best locations are for siding extensions and other line improvements.

Train Performance Studies and Computer Models

The GPS data can be used as a data source for train performance studies and associated computer models. Because we have the elevations of the track, grade can be derived. And because it is GPS elevation, it is all on one datum with no equations.

Once the data is in a database, signal spacing analysis can also be performed.

Processing of the Data

The contractor is responsible for processing the raw GPS data, integration of the inertial data, and constructing what is called the linkgeometry file. Events or features, like bridges and switches, are added to the linkgeometry file by the contractor using stereo imaging software.

The contractor ships to BNR the analog video tapes, the stereo images, and the linkgeometry file. BNR inspects the linkgeometry file to ensure that all events are recorded. This is done by reviewing the video. Random checks are performed on the feature coordinates by taking measurements with the stereo image software.

It is relatively easy to survey and process the data into a new data base. Since BNR already has many of the features surveyed in the RIS data base system, the process of linking these features and their attributes to the newly collected coordinate records in the linkgeometry file must be done. The linking is a semi-manual process aided by a computer application that searches the RIS database for likely candidates for matching. An operator makes the final decision about which records match, thus creating a link between a feature's position data and its attribute data. A new linkgeometry file is then created with an additional field that holds the record number of the feature's attribute record in RIS.

Once the linking is done, then a true GIS database is created. All sorts of questions about the features along a route can be answered either in report form or displayed graphically. Examples are: Display all the hot bearing detectors along a route. Display all the signals along a route along with their elevation and distance from each other for calculating braking distances.

Data File Format

The basic data file used since the beginning of the survey is called the linkgeometry file. It has undergone relatively few changes over the years. A railroad link extends from switch to switch. A switch is like a highway intersection, the route of the train can change at the switch. Track alignment points (TAP) are at least every 50 feet along the link. A TAP is simply a three dimensional coordinate in the center of the track. Features along the track link have three dimensional coordinates also. All records in the file are in the order you would see them if you drove down the track. Records for other tracks adjacent to the track surveyed are appended to the end of the file.

The following are the fields in the file:

Line Segment Number	A unique number for a route
Line number	A unique number for the track (switch to switch)
Facility Code	A three character identifier for the facility

Latitude
Longitude
GPS height
Geoid offset
UTMX
UTMY
Instantaneous Azimuth to the direction the vehicle is traveling
Instantaneous grade
Instantaneous curvature
Feet to the beginning of the link
Feet to the end of the link
Track number
Track type
Milepost
Unique identifier for the event
UTM zone
GPS time

Future Improvements

The biggest productivity boost for the mobile mapping portion of our survey will be when we can utilize broadcast differential services. Elimination of base stations will enable the vehicle to survey whenever conditions are right. It will also allow more flexibility in adjustments to the survey route schedule due to weather.

Unfortunately, current broadcast differential services do not provide the same level of accuracy of our current differential methods. The meter level accuracy is only 1 sigma and the vertical component is even fuzzier.

We feel an expansion of the Coast Guard's radio beacon system for broadcasting differential corrections is needed along our routes.

There do not appear to be any plans by the defense department to expand beyond the current number of Navstar satellites. There are only replacement proposals for the existing Block II satellites.

Ground based mobile mapping systems really need more observable satellites. There is some activity occurring in manufacturing receivers that can also receive the Russian GLONASS signals. GLONASS is operated by the Russian Military Space Forces with 21 satellites orbiting at 11,800 miles above the earth. Not much is known about the performance of this system, although the European community has shown some interest in promoting it as a competing system to the United States GPS system.

Helicopter mapping may provide another means to capture this data. A helicopter flies above all the obstructions encountered by ground based mobile mapping vehicles. Yet it can fly low enough to obtain high resolution data. Helicopter mapping systems in use today are equipped with GPS, scanning lasers, inertial systems, and video. Helicopter mapping is cheaper than aerial photography and may be more expensive than a ground based vehicle survey. But on routes with high traffic densities and/or obstructions to the GPS satellites, helicopter mapping may prove cheaper.

As a separate project, BNR measures clearances to adjacent objects such as tunnels, bridges, and retaining walls with a vehicle that travels the track. It may be feasible to integrate this operation with the GPS survey.

Once we have all of our system surveyed, we will need to perform "maintenance surveying" as changes occur. Most people do not realize the amount of track and facility changes that occur on a railroad. Just as your local highways always seem to be under construction, the same is true for many parts of our railroad. We need a way to resurvey a route, collecting only the data that reflects the changes. And then we need a way to move those changes into our systems with a minimal amount of effort.

Conclusions

BNSF is pleased overall with the current mobile mapping system configuration. It provides a minimal impact on our train operations and is capable of surveying many miles in a short amount of time.

Our experience though is that we collect data faster than we can process it. While a great deal of attention and effort is needed to make the field surveying a success, an equal or larger amount of attention and effort is needed to put the data into our systems.

The true worth of this project can not be realized until we can match the GPS data with the data we already have on our various track features. Once this is done, then we can really utilize the train performance models, the GIS software, and other capacity management and analysis tools.

The End.

BNSF'S ARGENTINE YARD PROJECT

By: R. L. Engle*

The Santa Fe Railway in recent years has recognized operating deficiencies in the present design of its Argentine Yard. Due to present operating practices such as running longer trains, and changes in traffic mixes and destinations we began looking at ways to improve the yard configuration for better switching operations. To better illustrate these operational problems, I would like to describe the present layout of the Argentine Yard and point out some of the "bottlenecks" as I go. You will note the concrete grain elevator in the background. This has now been demolished and will provide approximately 80,000 cubic yards of sub-ballast material for the construction of the new yard. Also the presence of the Interstate I-635 and the 42nd Street overpasses provided additional challenges in the design and track layouts.

The original Argentine Yard was constructed around 1906. The first classification yard, the westbound yard, was constructed in 1947. This schematic diagram shows the layout of the Argentine Yard. Highlighted is the westbound, in line receiving, classification and departure yard. The general design grade of the westbound yard is a .2% grade which is too steep to control cars for today's needs of coupling speeds less than 4 miles per hour. The car control is made up of an air operated master retarder and intermediate group retarders. Standard Santa Fe number 8 turnouts are used in the classification yard and number 10 turnouts are on the receiving and departure leads. The head end of the class yard has three specialty track work lap switches.

The westbound receiving yard is comprised of 11 tracks, with an average capacity of 66 cars. The classification yard is comprised of 8, 7 track groups for a total of 56 tracks with an average length of 34 cars. The westbound departure yard is 15 tracks, again with an average car capacity of 66 cars.

Figure 1. Existing Yard

*AVP—Construction, Burlington Lines, BNSF Corp.

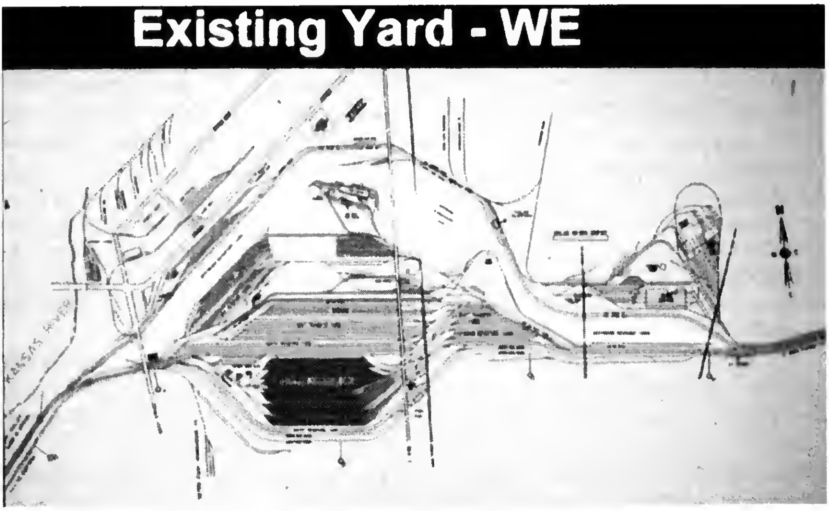

Figure 2. Existing Yard-WE

Figure 3. Existing WB Hump

The construction of the eastbound yard, shown highlighted in this schematic, began in 1969 and was completed in 1971 at an approximate cost of $17 million. The eastbound classifications yard uses an air operated master retarder and 6 group retarders for car control. The descending hump grade is –3% through the scale and –4.89% through the master retarder, and the yard body has a prevailing grade is –.1%. Inert retarders are used at the trim end of the classification for roll out prevention.

West Bound Yard

Yard	No. of Tracks	Average Car Capacity
Classification	56	34
Departure	15	66
Receiving	11	66

Figure 4. Westbound Desc.

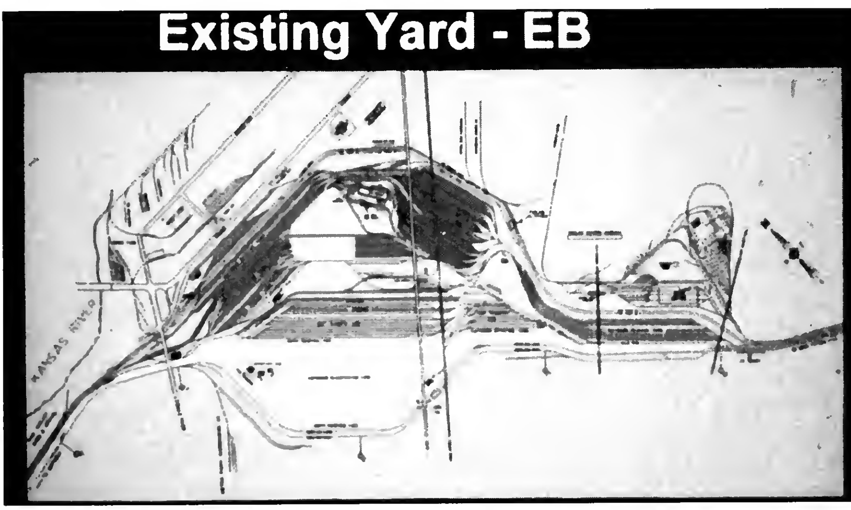

Figure 5. Existing Yard-EB

The eastbound classification yard is comprised of 6, 8 track groups, a total of 48 classification tracks with an average car capacity of 34 cars. The eastbound receiving has two wrap around yards on both the north and south sides of the class yard. There are 14 receiving tracks with an average 40 car capacity. The 22 track eastbound departure and transfer yard is "in line" having an average car capacity of 92 cars.

The local geology in the area of the eastbound classification yard with a very high water table provides very unstable sub-soils, creating a need for constant track maintenance.

East Bound Yard

Yard	No. of Tracks	Average Car Capacity
Classification	48	34
Departure	22	92
Receiving	14	40

Figure 6. Eastbound Desc.

Figure 7. Eastbound Mud

Argentine Yard's location in the Santa Fe System was very important. It is a major hub, and, in the merged BNSF environment, it will remain critical to the combined operation.

Today, an average of 44 trains move through Argentine daily, with more than twenty originating at Argentine Yard itself.

Santa Fe engineering and operating staff looked at several options. Since the Eastbound Classification yard was the newest yard, it made sense to save it and modify the remaining yards to accommodate our new operating requirements. One of the first proposals was to increase the capacity of the Eastbound Yard by adding one additional group of eight tracks to the north side of the existing yard and connect the Eastbound departure yard to the Westbound receiving yard. Another was to construct a new four track group on the South side of the classification yard and connect the Eastbound departure yard to the Westbound receiving yard. In both of the above studies, we concluded stabilization of the subgrade in the Eastbound classification yard as well as tangent point retarders were a must.

Using many ideas from Santa Fe's early studies and following a couple of design iterations, HDR provided a design concept acceptable to Santa Fe, to construct a new classification yard south of the existing westbound yard in the location where the grain elevator and eastbound yard now exist. Through detailed construction sequencing, the new plan was feasible and could be constructed while keeping most of the existing yard in service. However, due to the construction sequencing required, the construction span was estimated to be three years. Working with our operating team, it was finally determined that eastbound traffic could be switched temporarily at Emporia Kansas, with blocking at other strategic locations, thus closing down the eastbound yard and constructing the new yard with an estimated construction time of 18 months.

The new yard is to have a 10 track receiving yard. The shortest track is 6,100 feet and the longest receiving track is 8,500 feet, clearance point to clearance point. The 10 tracks average 7,700 feet in length.

The new departure yard is also 10 tracks with the shortest track at 5,000 feet and the longest track 8,000 feet. The average departure yard track length is 6,600 feet long, clearance point to clearance point.

Both the departure and receiving yard have running tracks used for movement of power to and from trains and to facilitate other yard moves. The design limited curvatures in these yards is 3 degrees or less to prevent by-pass draw bars at car coupling. The turnouts in the receiving and departure yards will also have electrically operated switch machines and will be controlled from the yard tower.

The new classification yard is a 60 track bowl comprised of 6–10 track groups. The average bowl track length is 2,666 feet with the shortest track 1,356 feet and the longest track 3,914 feet, clear distance. The switch leads, both head end and trim end are standard No. 8 turnouts, including 19 equilateral number 8 turnouts and one number 8/10 lap switch at the head end of the bowl. Air operated switch machines will be used on the head end of the new bowl, and electrically operated switch machines are to be installed on all trim end switches.

Figure 8. Yard Operations

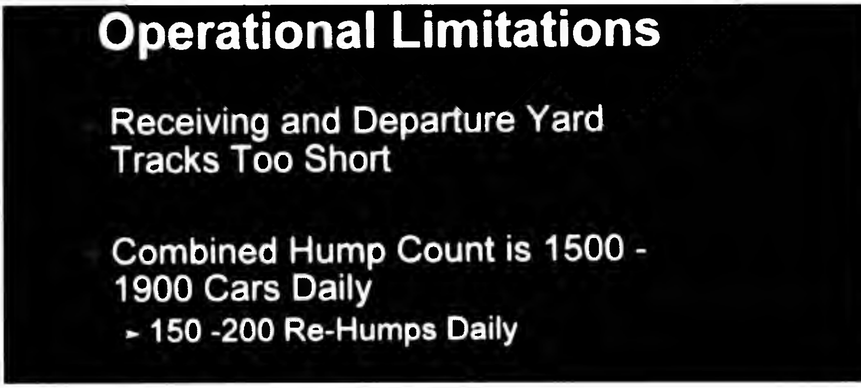

Figure 9. Operational Limitations

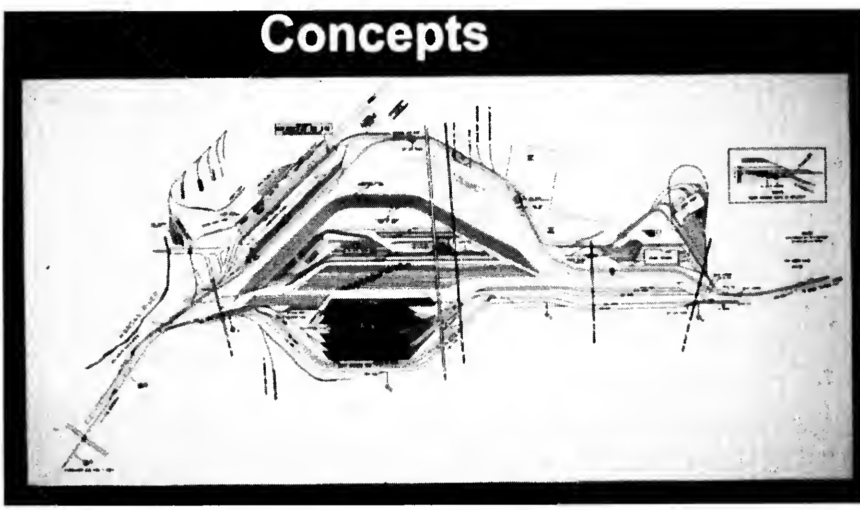

Figure 10. Concepts

The car control is comprised of a 24 air cylinder master retarder, 6–28 cylinder group retarders, and 5 cylinder tangent point retarders on one rail only. In addition, each classification track will have an average of 750 feet with distributive retarders installed. Our present estimates for car control is 98% of all cars going into the tracks to couple at a coupling speed of less than 4 miles per hour.

The crest grade approach is +3.09%. At the crest there is a +.49% grade, then the uncoupling area breaks away at a constant descending 3.04% grade across the weigh in motion scale, then through the master retarder at a –4.89% grade. There is a –.8% grade between the master and group retarders. A 0.0% grade connects the group and tangent point retarders. The exit speed from the tangent point retarders is targeted to be 4 miles per hour. The distributive retarders are to be placed on a descending 0.27% grade for an average of 750 feet per track. A –.08% grade then continues to the reverse grade to prevent rollout. At the trim end, a prevent roll out grade is a minimum of 300 feet of .4% grade. The design concept was to have at least a 1.2 foot of rise to prevent rollout and ideally it was planned to have 400 feet of .3% grade where it was feasible.

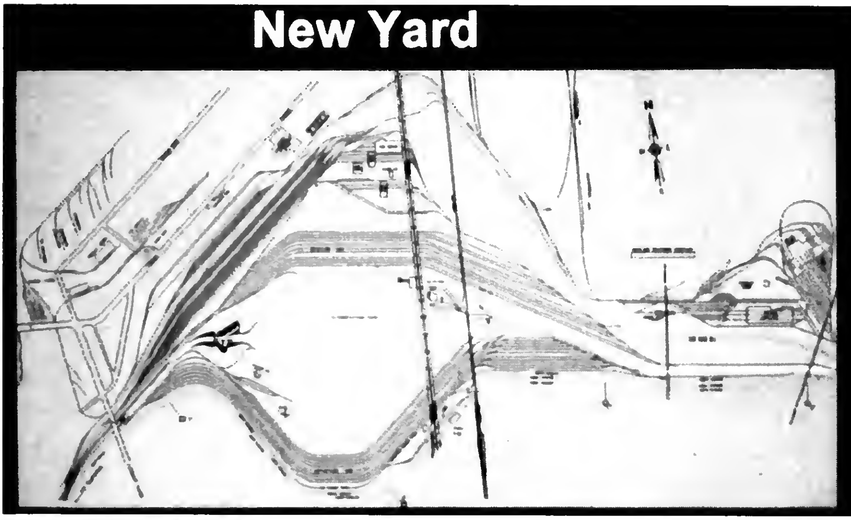

Figure 11. New Yard

Figure 12. New Yard—Receiving

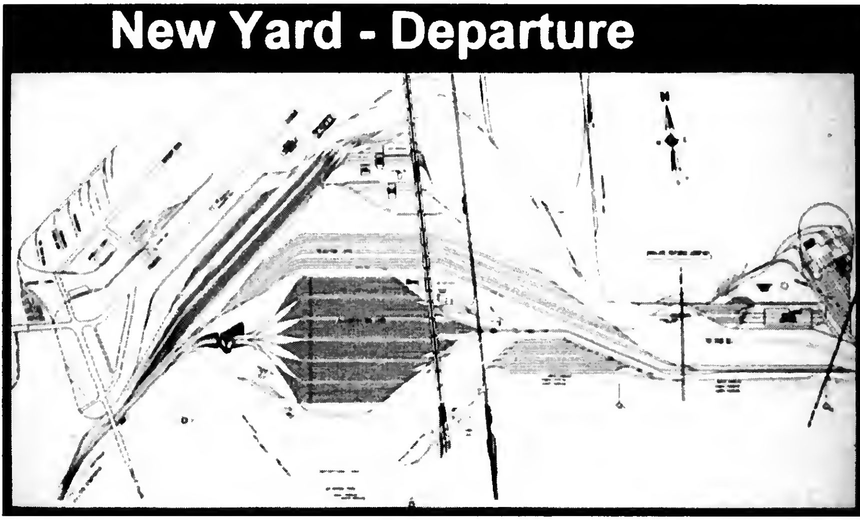

Figure 13. New Yard—Departure

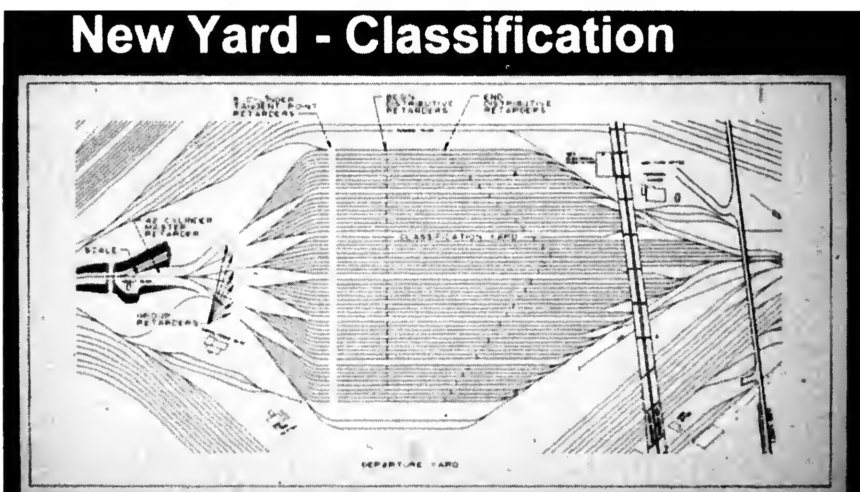

Figure 14. New Yard—Classification

The head end of the bowl has two pull back tracks with cross overs designed to allow a humping operation to proceed while another hump crew is pulling out of the receiving yard preparing to begin humping when the first crew is finished. The shortest hump pull back track is approximately 6,900 feet long and the longest is 9,300 feet.

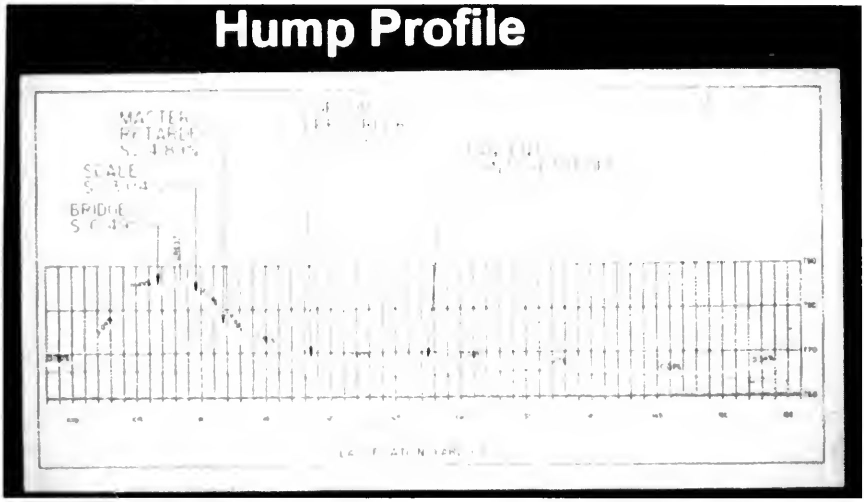

Figure 15. Hump Profile

Figure 16. Distributive Retarders

The trim leads are over 6,200 feet in the clear with cross overs and connecting tracks which would allow up to 4 trim engines to work at one time building trains. However, the design has intended that 2 trim engines could easily handle train make up at present and proposed yard through-put.

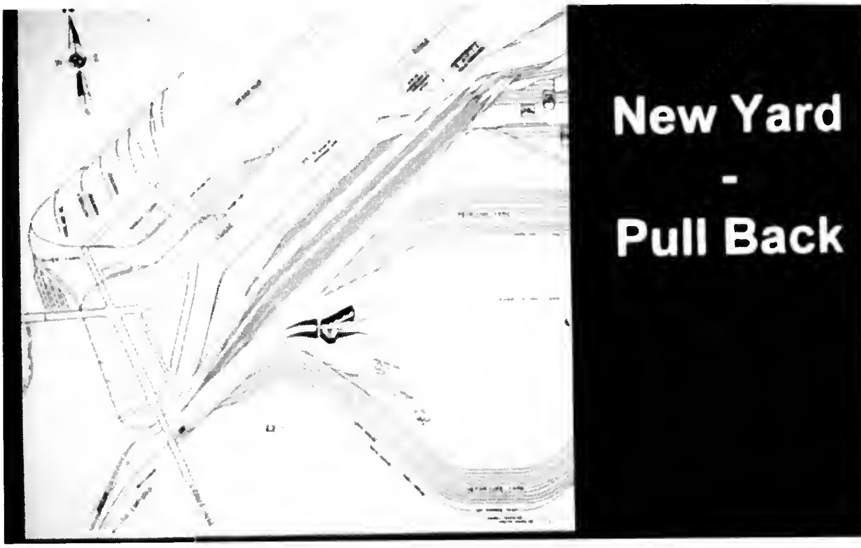

Figure 17. New Yard—Pull Back

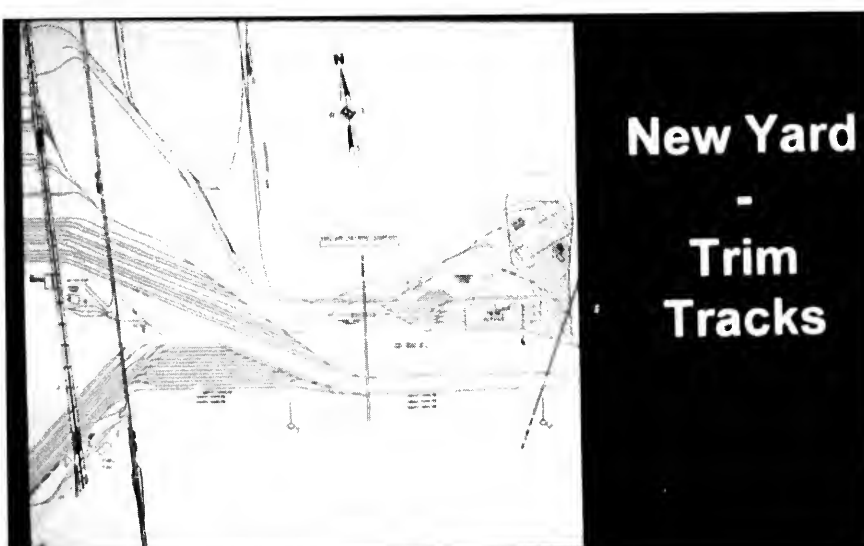

Figure 18. New Yard—Trim Tracks

The design also allows a close connection at the receiving yard, hump end and also at the trim end to allow less switch engine movement while making up trains. Train line air is to be installed in the departure yard near the lead ends to allow pre-charging on trains while awaiting power.

In addition to the new class, receiving and departure yards, an additional 3rd main line is to be constructed and the fueling facility expanded to allow fueling on the new main line.

Early on in the project, after the initial cost estimates were provided by HDR using all new track material, we went through an exercise of " value engineering" to reduce costs and get the biggest return on investment to justify the project. We decided to evaluate the condition of the track and switch materials in the yard and determine benefits and costs to reuse and rehabilitate existing track and turnout materials. Following inspections by our track people, we estimated reusing 60% of the existing ties. We reused all welded rail that was 115 pound or greater. Due to rehabilitating turnouts through component change, welding and grinding, only 33 new number 8's and 53 new number 10 turnouts out of a total of 248 turnouts will be required in the project. This concept and the derived savings gave the green light to our project.

Besides providing track design, coordinate geometry for the track, profiles and cross sections, HDR also provided design for improving the drainage system, structural design, electrical distribution and lighting design, utility and architectural design.

The drainage system is designed to drain surface water and sub-surface water through heavy duty poly-ethelene plastic perforated pipe. The new drain system is to tie into the existing main storm water system.

The new crest bridge is a concrete box girder of Santa Fe standard design set on H-pile and concrete backwalls.

The greatest structural challenge was the need for a retaining wall to separate the "high line" main line from a new main line running below. Due to varying soil conditions of the fill and a global soil instability problem at the site, the retaining wall is to be constructed of H-pile on 8 foot centers

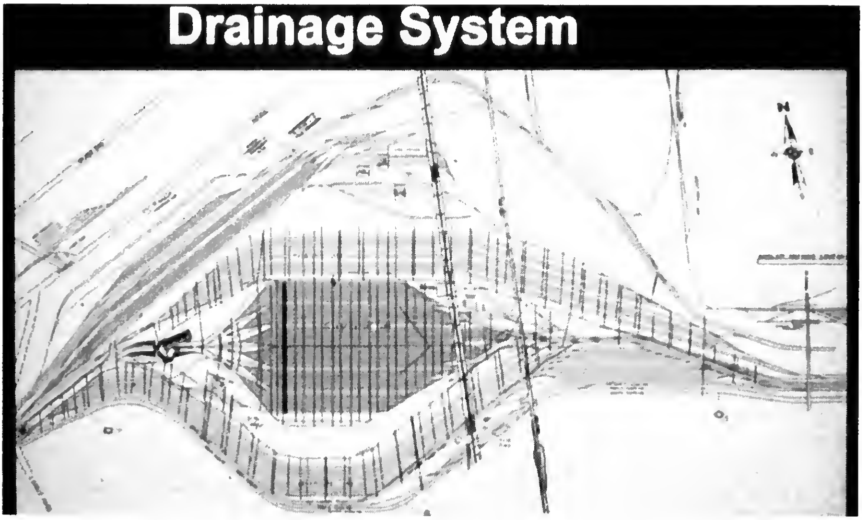

Figure 19. Drainage System

Figure 20. Retaining Wall

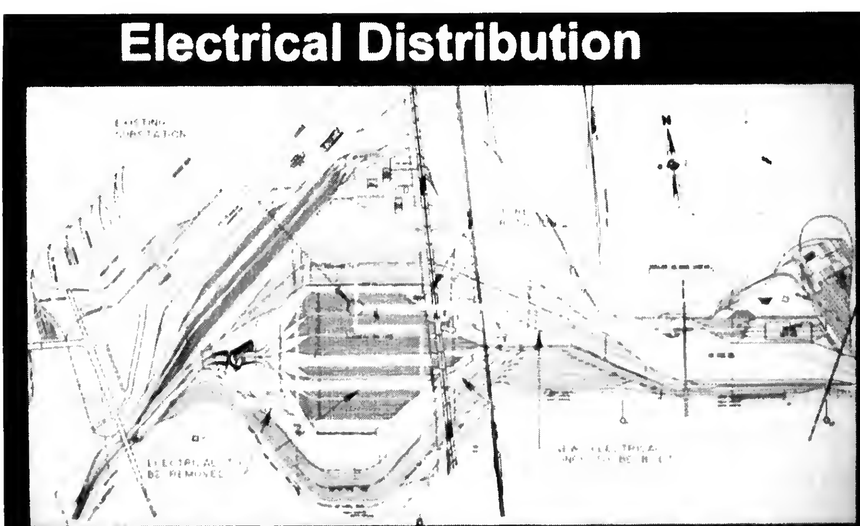

Figure 21. Electrical Distr.

and wooden lagging for the wall filler. To eliminate the instability problem, soil anchors or tie backs are being used and tied to each H-pile. The wall averages 7 feet in height for a length of over 1,400 feet and is to be concrete faced for long term durability and performance. A stop log gap is to be widened to allow for the new intermodal switching tail track at the yard's west end.

As luck would have it, a new electrical substation recently installed at the Argentine yard will require relocation. However, due to the redundancy we now have in the yard's electrical service, we were able to keep the yard's electrical service in operation and relocate the new switch gear to another location, out of the way of new yard construction providing considerable savings to the project. The new yard is to be lighted using 120 foot monotube light poles for the classification yard and pole lighting along the receiving and departure yards for night time inspections.

This figure shows the complexity of the utility design which included gas, sanitary sewer, air and water. Some of which was relocated, some renewed and some placed new.

Our plans are to use as many existing buildings in the yard as possible. By remodeling some buildings for their new use, we only required the design and construction of a small, one man building at the crest, a Train and Engine crew building combined with the computer and signal equipment area to operate the hump, and a small trim building for trim crews to work from.

The time line figure shows the activities of this project from the conceptual design, final design and construction. We started conceptual design with HDR in June 1994. Santa Fe approved the concept and began estimating and budgeting process in Sept. 1994. HDR was given the go ahead to begin final design in Oct. 1994. Due to the pending Burlington Northern Santa Fe merger, the project was placed on hold until July of 1995. HDR restarted final design, provided to Santa Fe 80% plans for review on Nov. 15, 1995. Bids were submitted to 6 contractors on Dec. 1, 1995.

As with all projects, there are notable efforts by individuals and I would like to specifically thank the following people for their expertise and exceptional efforts. From HDR, Andy Anderson provided his years of technical knowledge, combining his operating and engineering expertise to this project. Ron Poulsen provided additional design support and managed the design process to control

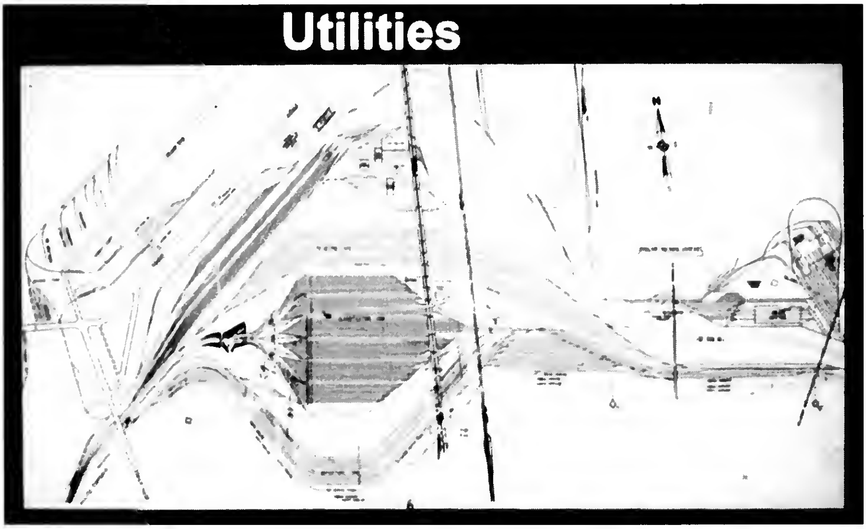

Figure 22. Utilities

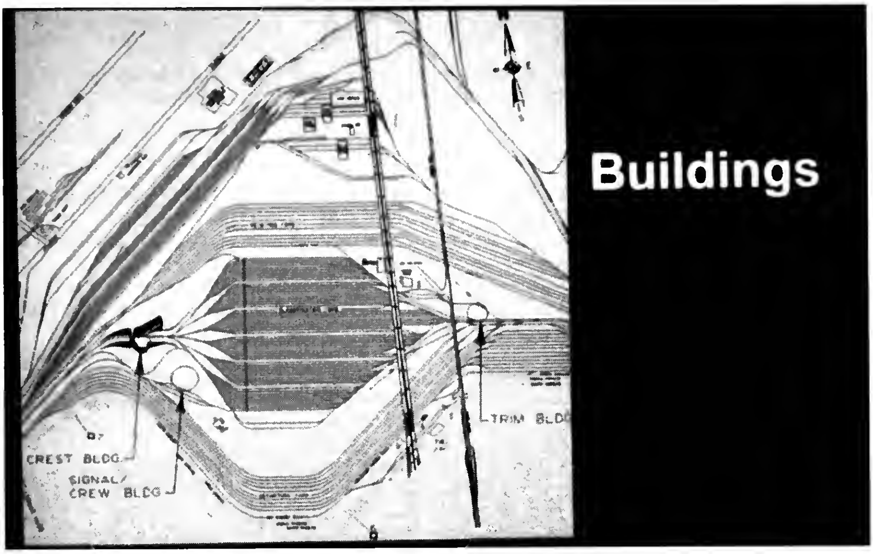

Figure 23. Buildings

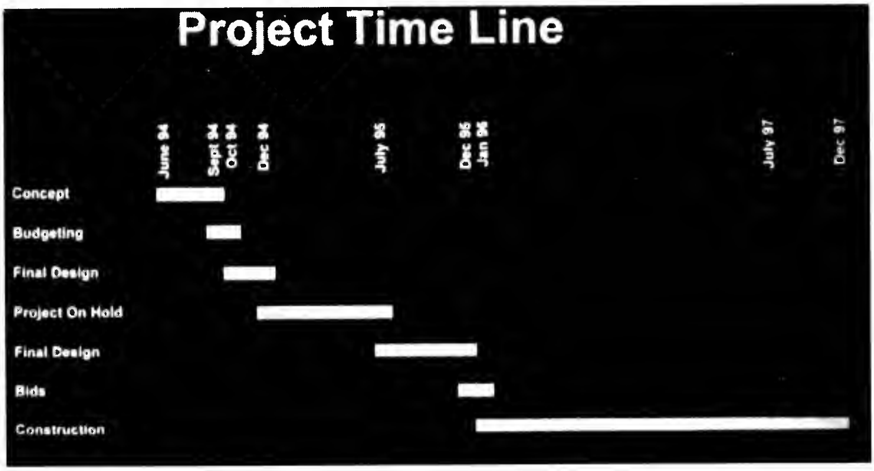

Figure 24. Time Line

Figure 25. Rendering of New Yard

costs to Santa Fe, ensuring the design was completed in our short time frame. Dave Dealy, Vice President-Transportation, BNSF, provided guidance and support for this project in seeing that operating requirements were met and budgeting was secured. John Prestridge, Project Engineer for BNSF has devoted himself to this project, and without his effort, we would not have made our goals.

If one were to visit Argentine Yard in Dec. of 1997 we believe this above photo enhancement accurately reflects what you would see.

ERGONOMICS AND MACHINE DESIGN

By: Carter Jones*

Today, I am going to discuss ergonomics as it relates to maintenance of way equipment; specifically, a look at ergonomics from a customer or end-user standpoint.

First, a confession. I must tell you that I am neither a designer, nor a safety engineer. But, ergonomically designed equipment is vitally important to me, to my railroad, and to all equipment customers. Our discussion here will be mainly about practical applications and not a design treatise.

The path we will follow today is one of definition, scope, challenges, examples of good ergonomic practices in actual application, and some recommendations for the future. As we follow this path, it is important to note that the incorporation of good ergonomic designs into equipment is a process, not a destination.

First, some obligatory definition and history. The word "ergonomics" comes from two Greek words: "ergo," meaning work, and "nomos," meaning laws. Therefore, ergonomics is the study of the laws governing work or the study of people in relation to their work. Ergonomics leads to human engineering, the real application of people and workplace solutions, and in our case today, people and machines. The science is fairly new, having been born out of matching men to war machines in England in 1940.

The purpose of ergonomics and human engineering is very straight forward; increasing the safety, health, productivity and job satisfaction of workers, in our context, through better machine design. In today's railroad environment, nothing attracts us quicker than opportunities for improved safety and lower cost. Thus, it is a timely subject and a classic type of engineering problem. Here, we are solving a boundary layer condition where the boundary layer is the person-to-machine interface.

So what are these conditions and how do they translate into design considerations? In a strict sense, the boundary conditions include the totality of the working environment and the person in it. Broken down further, we first look at the direct work surroundings, which includes the space, sound, light and vision, air quality, and vibration characteristics. We would then look at the expected worker conditions, including all the physical and psychological expectations, the demands of lifting and moving, and the frequency and speed of operations.

Ultimately, a good ergonomic design takes all these factors into account along with all the other design considerations, and creates machine environments and layouts which draw an operator in. Once in the loop, the operator performs work that is safe to himself and all surrounding him; work that is high volume and work that is of high quality.

I know, it sounds like selling soap; but, we are simply talking about good business. If the machines are designed right the first time, everybody wins. The customer and supplier each save money, through no additional, direct or indirect costs associated with re-design. Perhaps most importantly, the operator is safer, more productive, and more satisfied.

Maybe you feel the importance of an integrated design process is being over emphasized; but, as an industry, we have fragmented our approach to ergonomics. Typically, the customer will push the supplier on a specific topic such as seats, or specific controls, and in turn, the supplier will fix that specific problem. To make matters worse, sometimes that fix will be limited to that one customer.

Well, having done my preaching, I hope you will forgive me if I act a little fragmented today and slice up this topic to a more manageable size. Effective design clearly includes all of the aspects of ergonomics, as defined earlier; but I have neither the heart nor the expertise to discuss the

*Manager Maintenance of Way Equipment Operations, Union Pacific Railroad

psychological impact of operating equipment. And I sure don't have many pictures to show you that would speak to that topic. We are really not going to touch much on environmental factors either. We are going to focus on what most of us view as ergonomics, the physical layout of the workplace.

On a piece of equipment, the significant physical layout area is mainly, although not exclusively, at the operator's station. The basis for workplace layout starts with the topic of engineering anthropometry, or the study of the size and shape of the human body. This science is a static method of fitting people with their physical variances to work stations. People have been surveyed by being measured in 35 different ways, such as, standing height, seated height, and head length. The results are tabulated statistically. A designer then corrects for additional operator clothing and equipment and then tries to maximize the number of people his design will fit.

It seems complicated, but the tables are published. It's simply a matter of applying the knowledge. Certainly, many of the sub-suppliers do take advantage of this information, for example, seat suppliers, as do most of the equipment manufacturers in their cab and control designs.

A more complicated area is human/machine dynamics. Information on dynamic aspects of design is available, but I am not sure that it is being used to its fullest. Here, we are talking about movement and repetitiveness, often coupled with weight. In an industry where markets are limited and use is often intermittent, our challenge is to implement effective designs where information may not be readily available and returns are smaller. Human dynamics in relation to machine operation needs further study by all of us to avoid potential problems in the future.

We have done our definitions, theory, and preaching. In just a minute, we are going to move on to some of the "real world" examples. First, let's focus on the three main design processes: Integrated or built in design, Rebuilt, such as is done through a major shopping or recapitalization of equipment and After-The-Fact Solutions, or field fixes. After-The-Fact is the least desirable because it is more inconvenient and costly to implement; but it is important to recognize that often an After-The-Fact Solution fuels the other two processes. Here we are going to focus mainly on the Integrated Design and Rebuilt applications. Now let us see the real world, or "ergonomics in the metal," if you will.

One area that has been greatly improved in recent years is the process of placing ballast onto the track structure. At Union Pacific, we are proud of the efforts we have made; first, with powered ballast gates, and later with remote control ballast gates. With remote control, the ballast unloader is not only exerting less effort to unload the ballast, but is also removed from the immediate dust cloud surrounding the unloading.

On the more traditional side of maintenance of way equipment, improved operator seating, especially in machines with enclosed cabs, has been one of the most visible areas of ergonomic development. Today's seats offer adjustability as a key feature. Most of us, Union Pacific included, first started to practice ergonomics in our selection of seats.

Controls have been another area of development. Joysticks have been popular in many of the multi-tasking functions we want operators to do; however, joysticks have not been a panacea. On the market, there are other control devices which have not been used much in our industry, but may bear looking into; such as proximity devices. The advantage of a proximity device is that it can activate a function with very little movement.

Tampers have been an area of all kinds of improvements, including switch activated buggy systems. They are easy to operate, require less help, and are quicker to put in position than manually assembling the buggies. Some other tamper improvements are better steps and handrails. Operator access is very important. How many times do people go in and out of cabs during a day? A lot! Anything we can do to improve access helps.

Operator access and protection have been improved on all types of equipment. Access is important on all types of equipment. Railheaters, spike drivers, and anchor squeezers are just a few of the smaller machines where access improvements have been made. Although more of a safety issue than an ergonomic issue, we at Union Pacific use chainable 'Live Track' signs on all our operator driven equipment to warn the operator on the live track side of the machine. On brush cutters, of note are blade guards and truss frames to protect the cabs.

There are some simple things that can be of tremendous help. The laser cannons used on tampers require batteries which must be removed and recharged periodically. Locating the battery box in an easy place to get at makes a lot of difference to an operator. Establishing remote lockup linkages on turntables relieves the operator from manual lifting.

Other items are simple, yet require a paradigm shift. Changing the manner in which an operator uses a rail saw, simply keeping him in a more upright position, is good. Another simple concept is remote mounting filters. As with many in the industry, we specify remote mounted filters on many of our applications. In addition to making it safer, greater accessibility to filters means maintenance is more likely to be performed.

Another area of development we are proud of at Union Pacific are modifications made to Burro Cranes and their transport cars. Major work was done to apply additional handrails, a rear ladder and a folding catwalk attached to the car. This catwalk permits the operator to safely move around the machine, especially when he is tying it down. When work is completed, he folds the catwalk back up for safe transport. The changes on the Burro Crane and its car are the end result of a number of UP Quality Improvement Teams.

Another design, proposed by a Union Pacific Quality Improvement Team, is larger in scope. It is a proposal for lifting a 500-pound spare tire to the ground through semi-automatic means. This scenario fits one of the classic design models. The folks out on the gang saw a need to be able to safely change a crane spare tire. They took their idea and design to our shop people, who modified it and made it available to the field. We then placed it an an option on our specification. We went from need to prototype to production model to fully integrated design.

We have had some theory, and have seen some of the practical results. What are the processes to keep improving designs ergonomically? The processes are not different from those already needed to improve machine design.

1. Be proactive. Let's not allow government agencies to drag us kicking and screaming into the present.
2. Recognize and treat design as a collaborative process.
3. In line with Item #2, allow feedback from end-users. They identify a lot of problems. On Union Pacific, we have a formal feedback process to permit improvement to equipment. Many of the specification changes we make come from machine operators.
4. Stay involved with industry organizations. A lot of good support comes from AREA. An "Ergonomics in Design" seminar was held after a Committee 27 meeting a couple of years ago.
5. Stay abreast of regulations and potential regulation changes.
6. Know where to find and use ergonomic reference material. Most of the source material for equipment design is not originally generated within the railroad industry.

When we look at the safety and financial issues, using good ergonomic practices in design is not really an option.

Our main choice is whether to be proactive or to be pulled along. At Union Pacific, we have chosen to be proactive. We hope you join with us.

EXTENDING MAIN LINE TRACK CENTERS

By: D. L. Deterding*

About 130 years ago, General Grenville Dodge and his pioneering crews first walked down this Platte River valley.

They were surveying a route for Union Pacific's transcontinental railroad, and no doubt believed that one set of tracks . . . a single-line railroad . . . was all that would ever be needed.

But when the line was completed in 1869, there was a lot of traffic, right from the start. Imagine . . . six trains a day! One passenger and two freights each way, each day!

It was heavy and fast traffic, too. The typical freight was 22-cars long . . . and you could get from Council Bluffs to Promontory Point in just four days . . . and from the Point back to Council Bluffs in only five or six

General Dodge must have been pleased, as well he should have been. But he would be amazed to see what has happened to his railroad in the twentieth century. By 1909, the traffic was up to 22 trains a day. Union Pacific had to become a double-track railroad. General Dodge's line became our No. 1 Track on the North . . . and a new No. 2 Track was added on the South.

This worked well . . . and is still working. But traffic is also still increasing. Railroads are regaining significant shares of the cargo they started losing to trucks thirty or so years ago . . . and coal traffic out of the Powder River Basin has doubled in the past five years.

Our double-line track between North Platte and Gibbon, Nebraska, has become the heaviest haul freight line in the world. Right now, we're up to 120 trains a day over this 108-mile stretch . . . and we expect more. So Union Pacific is doing something exciting that most railroaders ever imagined would be necessary . . . building a third, and in one case a fourth . . . mainline track.

I am in charge of construction for that third line of mainline track. And today, I want to tell you how we built the first 16-miles last Spring.

Basically, all trains on our East-West system use the North Platte to Gibbon Corridor. Gibbon is where our main line connects to the Marysville Sub, so East of Gibbon, the traffic decreases substantially.

In 1994, the North, or No. 1 Track, which carries pre-dominantly Westbound trains . . . carried 81.4 MGT, or million gross tons. And the No. 2 Track, primarily for Eastbound traffic, carried 169.1 MGT. In 1995, No. 1 Track increased to 90.73 MGT and No. 2 Track to 188.46 MGT.

As you can imagine, with this quantity of traffic, it is very difficult to find time to perform adequate maintenance. My Gang which supports the New Construction is frequently in the clear for a train per FRA regulations.

The entire segment from North Platte to Gibbon has Universal No. 20 cross-overs approximately 7 miles apart. The segment is also under CTC . . . Centralized Traffic Control . . . with all movement controlled by the Harriman Dispatching Center in Omaha.

Even with crossovers installed this close together, and with CTC, adequate track time for maintenance crews was just not attainable.

The decision was made to build a third line. The configuration of how this track was to be built was an important part of the decision. No. 1 and No. 2 tracks were, you know, track centers at 13-feet and is certainly an outdated standard. It prohibits wide loads, and more essentially, interferes with track maintenance programs . . . especially, Tamper's P811 track renewal machine, which restricts train movements on adjacent tracks.

*Director-Construction, UP RR

Figure 1

To eliminate this problem during maintenance, it was decided that No. 3 track would be built at 27-foot centers from No. 2 track . . . or 40-foot centers from No. 1 track.

Our standards for track renewal on this corridor require the use of concrete ties and head-hardened rail. With the price of timber these days, concrete ties are not only better, but cost less per track foot.

We use Tamper's P811 track replacement machine to do a complete renewal, with 8-foot, 6-inch concrete ties and 133-pound rail. Use of the P811 on 13-foot track center territory requires the P811 operation to be shut down when a train is passing, which greatly reduces productivity.

But the other alternative of totally shutting down the remaining single track making adequate work windows on this busy corridor would delay trains far too much.

Constructing the new No. 3 track first, at 27-foot centers from No. 2, allows us to open No. 3, then relocate and rebuild No. 2 at 20-foot centers from each outside track.

Why 20-foot track centers? There are several reasons. I'll give you just two. Number One, standard switch-tie layouts can be used at crossover locations which facilitate maintenance. And Number Two, track maintenance equipment with enclosed cabs can continue working with a train on the adjacent track, which greatly increases production.

For the first 14 miles, a contract was let to Neosho Construction of Topeka, Kansas, to build the new grade for No. 3 track and to extend the culvert structures. The additional bridge work was done by our own UP Gangs.

The grade work was basically routine, except for a problem with cinders. Along No. 1, our original mainline, cinders were still there from the steam days. When cinders are dry, they are a fine dark gray powder. When they are wet, they turn to soup. Either way, it's not a good base for a modem railroad.

Where cinders were encountered, we sub-excavated to a minimum of 2-feet below finished subgrade. And, if less than desirable material was found, fabric was installed to provide additional support and also act as a medium to separate materials. A well-graded sand and gravel was placed on the fabric to subgrade elevation. There is plenty of sand here, because this material occurs naturally in the Platte River Valley.

The next challenge was sub-ballast. There is no high-quality material nearby, so this had to be imported from our granite ballast pit about 250-miles West, near Cheyenne, Wyoming. Sub-ballast was delivered to the contractor in unit gondola and air-dump trains. Large backhoes were used to unload the gondolas and air dumps. The backhoe unloading an air dump is much more expedient than using air, taking less than one minute per car, and the operation can go year-round.

If the material is frozen, it can be done in less than two minutes per car. One machine, (in this case a Cat 966 front end loader) holds the car down while a Cat 235 backhoe rakes out the material. A No. 14 grader then removes the material in front of the door and the cars sit back down. The cars hold about 40 cubic yards, and 2,176 carloads of sub-ballast alone were required on our first 16 miles. New service roads along the entire length of the job were also capped with sub-ballast.

One of the main objectives on any track building job is to close as many grade crossings as possible. Fortunately, we had 200-feet of right of way in many areas, so we were able to close several farm crossings and build parallel access roads out at the ROW line to take their place. These were also surfaced with 3-inches of the same sub-ballast we were using on our new grade.

The first 14-mile section from Mile Posts 267 to 281 left a two-mile stretch of double track to the Bailey Yard in North Platte. This was let as a separate contract due to a major obstacle . . . the Platte River.

The existing bridge was 1,000-feet long, consisting of twenty 50-foot ballast deck plate girder spans. A hydrology study showed we could eliminate two spans.

A contract was let to D.H. Blattner and Sons of Avon, Minnesota, to construct a new 900-foot double track bridge.

The bridge was designed and built to accommodate a fourth track for sometime in the future. For now, we're using the extra width for a maintenance road.

The bridge substructure consists of concrete piers, supported on 14 HP89 pilings, driven 80 feet. The superstructure girders were 50-foot precast, prestressed double-cell boxes manufactured at Wilson Concrete in Omaha. These were placed five wide to form the new deck. The interior girders, weighing 54 tons, were hauled from Omaha by truck, but the 18 outside girders with curbs were too heavy and had to be hauled by rail.

While the bridge was still under construction, the 14 miles of grade to the East was completed, and track construction was initiated. A temporary No. 20 turnout had been installed at M.P. 281 and a short setout track was built for unloading materials such as sub-ballast and ties.

Construction commenced East from Mile Post 281. Between Mile Posts 276 and 274, a siding called Keith had been constructed in 1979 to mainline standards, but not at the right track centers. For the current project, it was completely removed and a new grade built for the new third main line.

The wood ties removed from this siding were left preplated by removing only the outside rail spikes . . . then moved to the North Platte Yard along with the rail to construct Numbers 7 and 8 tracks at our 8 track Eastbound fueling facility. By leaving them preplated, Tamper's new machine could be used to construct the fueling tracks.

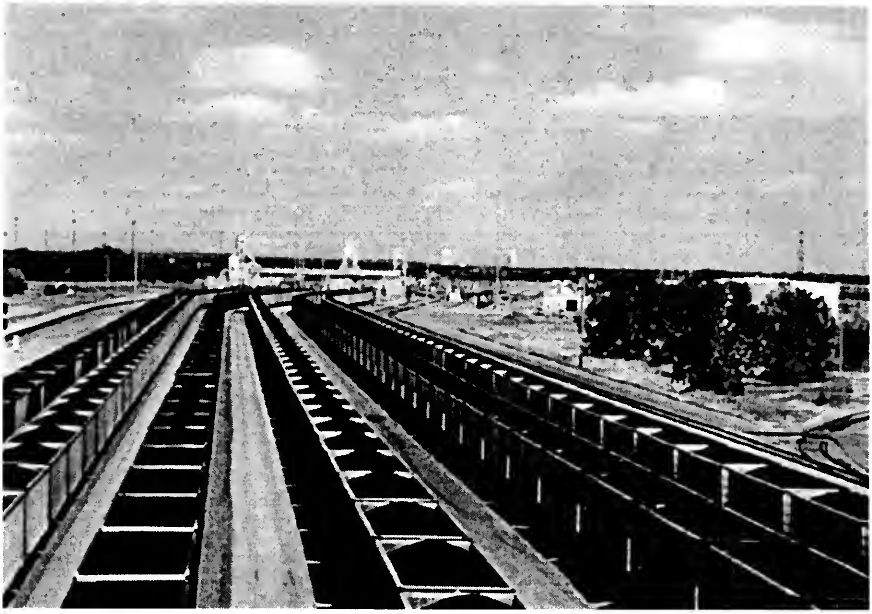

Figure 2

At each end of Keith was a one-way crossover. This design was left "as-is," with an additional switch installed in the new No. 3 track to complete a crossover. This switch layout will accommodate simultaneous moves from No. 1 to No. 2 and from No. 2 to No. 3.

The new switches at this location are No. 20 concrete premium turnouts with movable point frog, good for 40 MPH. These switches were constructed in panels, and placed using a 125-ton crane supplied by the contractor.

The new No. 3 track was constructed to M.P. 267, where a No. 30 turnout was installed. Because of the heavy loads on No. 3 track, due to Eastbound loaded coal trains, it was constructed with a turnout side to No. 2 track. This turnout is good for 60 MPH, so turnout trains won't be delayed.

After final surfacing and lining, and welding all joints, this portion of the new track was placed in service just over a year ago, on April 27, 1995. At the same time, the new Platte River bridge was nearing completion . . . just waiting for girder delivery.

Now we get to the most challenging part of the job . . . relocating and reconstructing No. 2 track. This required a considerable amount of equipment from both the contractor and the Union Pacific. D.H. Blattner and Sons, the bridge contractor, was awarded the job . . . which included the grading to complete the two miles from M.P. 281 to M.P. 283. Blattner has first-rate equipment and excellent personnel, and did an outstanding job of fulfilling our requirements.

Working between two live tracks was a real challenge to maintain production and keep the trains moving, but safety was always the primary concern. As you know, Union Pacific uses a Form B track bulletin to protect our construction and maintenance work on track projects. This Form B ended up being 16 miles long to protect the reconstruction of No. 2 track, the construction of the new bridge and the grade being built from M.P. 281 to M.P. 283.

It was almost a nightmare trying to protect approximately 50 machines working through the limits of the job, and make sure they were all in the clear, when trains passed. With the quantity of trains on this corridor, we basically had a train cleared through our work limits on No. 1 or 3 track, or we had trains within our work limits continuously throughout our working hours.

It was impractical to shut down work and clear trains through. We started using 10-hour days, and we were experiencing train counts in the low 60's in just 10 hours. To increase production, we went to 12-hour days. Now, our train counts were in the middle 60's to a high of 76 trains in 12 hours.

To clear the trains through, we used a series of flagmen who reported to a main flagman. He cleared the trains at restricted speed not exceeding 20 MPH so they would be prepared to stop short of obstruction. All our enclosed cab machines as well as the contractor's machines had radios, as good communication was a must.

The location had good site distance and only one, 1-degree curve. And for added safety, we set things up so that any equipment swinging across a live track was always facing on-coming trains.

The first task was to remove the No. 2 track. The rail had only 602 MGT and was to be reused. First, a tie-marker went through and marked all ties that were to be salvaged. A System curve gang then removed all anchors and pulled all spikes on the ties that were not kept . . . but only the outside rail spikes on the ties to be retained.

The rail was threaded out to the No. 1 and No. 3 track shoulders, then all remaining plates, anchors and spikes were picked up and placed to the North side of No. 1 track to be loaded into dump trucks and hauled into the Yard.

The contractor had the responsibility to remove the ties. All that were to be kept were bundled in 5 x 5 bundles with 2 x 4 spacers on the plates, for use in the North Platte Yard to construct 7 run-through tracks for empty coal trains, and a new 17-track spare yard for coal cars. The rejects were disposed of by the contractor. We kept about 29,000 ties on this project, for a reuse rate of 60-percent.

We had a real concern to provide a good base for No. 2 when just changing centers 7-feet. The procedure we used was to windrow the best of the ballast section on each side. Then, approximately another one foot of material was removed and placed on the South side of No. 3 track.

This was mostly ballast with granular material and cinders mixed in, and was used to construct a service road on the South side. The goal was to remove enough material so that when the best ballast was windrowed back in, it would be equal to one foot of sub-ballast to get the new track to the same elevation of No. 1 and No. 3 tracks.

To compact the sub-ballast section, 50-thousand pound vibrating rollers were specified. If there is a weak subgrade, the vibrating rollers will find it, and find it they did. It sure didn't help that we were doing this project in the Spring. To remedy the problem, we placed a 12-foot width of woven fabric the entire length of the job, underneath the sub-ballast.

Maybe you saw the issue of TRAINS Magazine last November. It did a good job of describing how we laid the new concrete ties. We used specially-modified flatcars, each holding 168 concrete ties. An overhead gantry traveled the length of the cars, and as each 720-pound tie went down the conveyor belt, rubber pads were put on the tie. When everything was working right, we were able to lay 12 ties a minute on a 2-foot spacing.

Ballast, surfacing and lining followed, using 1,400 100-ton hopper cars. We had the work train supplying tie cars to the construction machine and ballast switched at night, so as not to delay progress in the day. It took just six weeks from the start of No. 2 track removal, grading for the new track and completion of new No. 2 track.

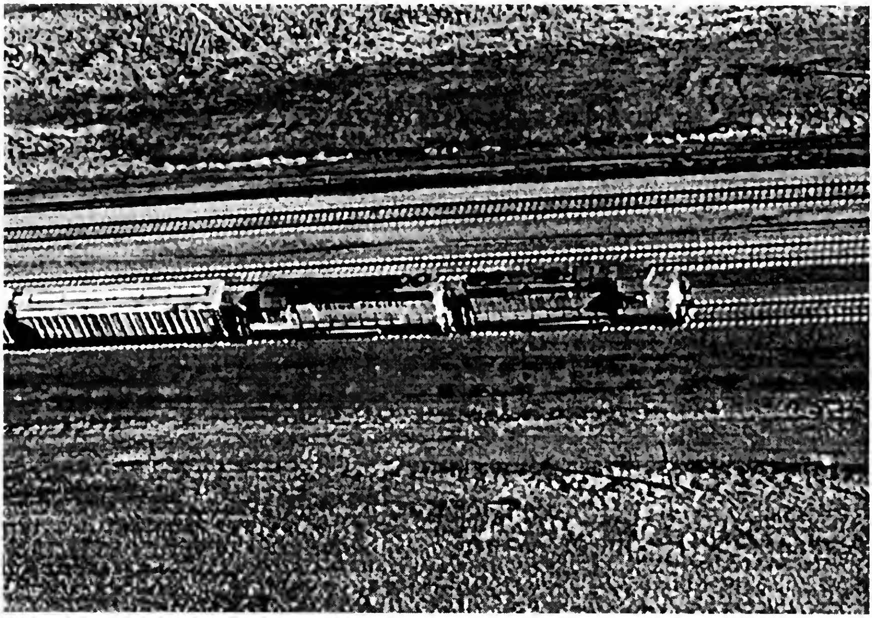

Figure 3

All design for the entire project, except for the bridge design, was done in-house, by Union Pacific personnel, and no unusual track permit problems were encountered.

The bridge was designed by H.N.T.B., of Kansas City. We had a lot of great effort and coordination by different departments of the railroad, as well as by the contractor. And I'm happy to report that the entire job was completed with no personal injuries. The only "casualty" was one dented truck fender, which lost a battle with a snowplow on a locomotive.

When work on No. 2 was finished, we constructed the new No. 3 track across the bridge, from M.P. 281 to M.P. 283. After this section was put in service, the No. 1 track was renewed using the P811.

While all of this was happening, we were also completing the final stretch of a quadruple track on the West side of North Platte. We now have four tracks for 10 miles, from North Platte to O'Fallons, where our North Plate Subdivision heads Northwest towards Powder River country.

So right now, we have 16 miles of new triple track with concrete ties, running East from North Platte. It's a beautiful sight, and it's just a start. Our capacity budget for 1966 has $68-million earmarked for more track additions in the North Platte vicinity, including another 19 miles of triple track to be constructed on the Gibbon end. And the plans are to keep going until we have a new three-lane super rail highway stretching the entire 108 miles between North Platte and Gibbon.

These new capacity projects will go a long way to making the Union Pacific even more productive, and to helping expedite trains in and out of the Bailey Yard . . . which, incidentally, is now officially in the Guinness Book of Records as the largest railroad yard in the world.

Thank you.

A.R.E.A. COMMITTEE 7
CURRENT STATUS AND UPDATE—
DEVELOPMENT IN TIMBER BRIDGES

By: Donald L. McCammon, Chairman, Committee 7*

Thank you for the introduction. A.R.E.A. Committee 7, Timber Structures, is in the process of completing several major tasks and finds itself embarking on new tasks to continue serving the railway industry. This is a brief report on our current status, research programs that are underway and our future plans.

Just a reminder, we are talking timber bridges. All those who do not have to deal with Timber structures or decks may find this a little boring.

We will show our current membership and breakdown by member classification. Our proposed manual changes, recently approved by the Committee, will be presented. With Board approval, they will be published in 1997. We are closely monitoring research programs being conducted by the AAR and have made suggestions for future timber bridge research. The research results may create future changes in our Recommended Practice. In addition, the recent computerization of the Manual caused a thorough review of the contents of our Chapter. We identified several sections that need updating.

We currently have 36 members with a slight majority being active railway personnel. Member Emeritus status has been conferred on three members who have given much to this Committee. Our Associate Members are equally split between suppliers and contractor representatives. Please note that what member classification we use may not be an accurate "Constitutional" classification. Hopefully the effort to clarify the AREA's Constitution will result in a better understanding.

We are working on finalizing several Manual additions and revisions that, with Board approval will be included in the 1997 Update. Recommended Practice for Design of Timber Bridge Ties, Design Tables for various length timber spans for E-80 loading and updated Pile Load Distribution Tables have been passed by Committee and are being placed in final format. Due to the computerization of the Manual, we did not know the final format until after our last Committee meeting.

Recommended Practice has also been developed for a new timber span and deck called Stress Laminated. This span and deck type has previously been discussed before A.R.E.A. by Shakoor Upahl, our Committee Vice Chairman. Willie Benton has been spearheading the effort. A typical section is comprised of laminations which are made from common 2 by material that is treated and then squeezed together by transverse rods. The first rail bridge was placed in service near Winnipeg in Canada. Several others are planned to be placed in service in the U.S. by additional railroads this year.

Our Committee previously talked to you about the use of exotic woods such as Azobe. We saw some initial interest due to their excellent decay resistance and strength properties with several timber decks, stringer and some ties for track uses being installed. However, we have not seen this interest continuing for some of the following reasons:

- Native woods continue to work well
- Native woods are cost effective
  Glued Laminated Stringers are being used for example
- It was difficult to work with the exotic timbers
  Drilling, fabrication, shipping and purchasing the timbers was not an easy process.

*Dir. of Rail Services, HNTB Corp.

We completed our assignment to revise our section on Timber Bridge Upgrading and Life Extension. For example, we now include that member addition and adding new members with greater capacity are recommended practice. We also discuss open deck to ballast deck conversion as well as composite bridges; timber pile, concrete cap, steel, timber or concrete spans. We now show several maintenance practices that have been in use. Examples are our recommended pile posting and pile restoration procedures.

Committee 7 is watching AAR's current Timber Bridge Testing Program and has provided input into its development. Currently the AAR is researching:

- Full size timber stringers to develop shear and static bending stresses by loading new and used stringers.
- Load path testing using AAR's TLV with its known loads to determine how the structure reacts.
- Effectiveness of timber structure upgrades to determine what happens when changes are made.
- Creation of a finite element analysis, a computer model, that can be used to predict structure response.

The testing began in 1995 and is continuing this year. The research conducted by the AAR is part of an ongoing effort to look at design guidelines for timber strength under heavy axle loading. The members of Committee reaffirmed their support for this testing program at its latest meeting.

The goal of full size timber testing is to determine shear and bending stresses and then compare to the existing allowable stresses. This may sound redundant as we already publish stresses in our Chapter. However, these allowable stresses are based on testing performed on perfect, small, clear samples of wood which are 2 inches by 2 inches in size. Actual timbers have splits, shakes, knots and other defects that do not match the test sample. We use factors to derive the allowable stresses. The factors used may be too conservative. To wake everyone up at one of our committee meetings, all you have to do is start a discussion on horizontal shear and you will not have to worry about the meeting being quiet.

This slide (Figure 1) shows the testing of a full size timber stringer being performed at the University of Illinois, Champaign. You can see the testing apparatus as well as the gages. During our visit we were able to observe a horizontal shear failure as well as a bending failure. Several of our more senior committee members expressed the observation that this was the first time in all the years on the Committee that they had actually seen a timber broken. We were able to watch the stress strain curves while the tests were conducted. Timbers have been donated by the Norfolk Southern and Southern Pacific. More used timbers are needed, but in particular new stringers are needed to complete the testing program—see Duane Otter of the AAR if you can provide some material.

Load path testing is being performed to verify how loading is transferred through the structure. Our Chapter has design factor, tables and formulas which may need updating. This work was performed on several UP bridges near Fort Collins, Colorado and a bridge near the FAST facility at Pueblo. The research is being performed by Colorado State University.

The effectiveness of structure upgrades, replacing timber stringers with Glued-laminated timbers, replacing timber open decks with ballast decks and adding additional stringers are being studied by the AAR on Southern Pacific structures in Texas. Research is being performed by the University of Iowa. What is particularly significant about this testing is that the line these structures are located on carries 70 MPH, 125 ton equivalent load, double stack traffic.

This slide (Figure 2) shows the testing equipment and gages on a SP bridge at Cline, Texas. A work train with weighed cars as well as actual trains were monitored. On this line several bridges were having steel ballast pan decks installed to replace timber open decks. Loads and resulting structure reactions were monitored before the decks were placed. The steel deck pans are used in areas where a track raise would create problems. At other locations, a timber ballast deck may be used. Another upgrade strategy used by the Southern Pacific is replacing the stringers with Glued-laminated stringers.

Figure 1. Testing full size timber stringer at Univ. of Illinois, Champaign.

Figure 2. Instrumental SPRR Timber Bridge at Cline, Texas.

The finite element analysis will allow us to gain a better understanding of the rail load distribution on the superstructure and prepare a computer model that is verified against the filed data accumulated during the testing program. The model is planned to be created by a Russian railway engineer working with the AAR this year.

Our committee has suggested that the following future research is needed:

- Small samples should be taken from the full size timbers tested and testing matching the small, clear sample testing be performed to allow direct comparison of stresses.
- Impact, due to the nature of timber as a material, is not used directly in our design calculations. We are thinking that the repetitive heavy loading and speed may be getting to a point where "fatigue" and impact may be causing problems. The question is raised "If we maximize the use of the material capacity are we creating the need to use other factors in our design?"
- Non destructive testing may be a means of assisting inspectors to provide quantitative information on bridge load carrying capabilities. What equipment exists and what can be developed to meet the railway industry needs?

We identified several items in our Chapter that needed modifying or updating during our review caused by the Manual computerization. For example:

- We still recommend calculating longitudinal force at a point six feet above the rail. Chapter 8 and 15 apply this force at the rail.
- We still recommend using hot creosote as a preservative to be used in the field for cuts or abrasions as well as pile cut off treatment.
- Our references to timber handbooks, grading rules and other resources are not current. In some cases, timber grading associations have published revised grading rules 2 or 3 times since our reference was noted.

Concern has been expressed on open deck bridges - tying inspection with track geometry. In our manual we assume all members carry loads equally. Due to horizontal shear failures or other problems that may weaken individual stringers, this may not be the case.

The results of the AAR Timber Bridge Research will need to be incorporated in the Manual. This will ensure, as AREA Executive Director Dave Staplin expressed, that we do not create two industry standards. We also need to look at our design and rating parameters in our general guidelines.

You realize that there may be some problems with your manual format, when a consultant calls you up and asks how timber bridges are designed as he is having difficulty identifying the procedures in the Manual. I challenge those of you here today to contact your Committee 7 member or myself and let us know your suggestions for improving our Chapter.

Timber structures are being used on both heavy axle, high speed lines as well as low speed branch lines. Timber bridges are also not just a Class 1 railroad structure. Our Committee recommends practices for timber structures. As you can see, we have a lot of work ahead of us. We need help from the users of timber to ensure that the Practices we recommend are the best, reflect current industry needs and are helpful to the railroad industry.

We are in the process of planning our next meeting. It has been proposed to be in San Antonio in June or July. Ken Wammel with the Southern Pacific has offered to let us observe some of the timber bridge work occurring in this area. Please contact me if you are interested in attending or joining the Committee.

I would like to thank Duane Otter of the AAR, Ken Wammel of the Southern Pacific and Dave Griffin of the Union Pacific for allowing the use of some of their slides. In particular I would like to thank the members of Committee 7 for their efforts and assistance. We have enjoyed serving the industry needs in the past and look forward to serving you in the future.

OPENING REMARKS AT THE 1996 AREA REGIONAL CONFERENCE, MEXICO CITY, MEXICO

By: Lic. Luis Antonio De Pablo Serna*

Mr. Jim Beran, President of the AREA, Mr. Edwin Harper, President & CEO of the AAR, Distinguished members of the AREA Board of Directors, Ladies & Gentlemen:

Welcome to Mexico!

First of all, I would like to thank you for the kind invitation to be a participant at the opening of the AREA 1996 Fall Technical Conference.

It is an honor, indeed, to be a participant in yet one more of these many professional, high quality, technical sessions that AREA has organized throughout the years since the formation of the Association almost a century ago.

The consistency and efficiency with which this Association operates is a good precedent to the North American Free Trade Agreement (NAFTA). Even more encouraging yet, is to see the benefits that the efforts of the AREA produce in the economy of each of the 3 countries involved, as well as the region as a whole.

The fact that Canada, the US and Mexico are able, today, to move and exchange large volumes of products using railroads, is in great measure, due to the compatibility of the railway infrastructure, and this compatibility is a result, in great part, of the hard work of this Association.

This year, the AREA conference takes place here in Mexico at a crucial time in the Mexican economy, with the re-structuring of the Mexican Railways. This restructuring represents a process of revitalization of the Mexican railway industry. Promoted by President Zedillo, the process has some similarities to the one that occurred in the US about a decade ago, and is now happening in Canada as well.

The program in motion is based on the participation of private investment, in the country's main railway lines. These have been subdivided into regional railroads to encourage healthy competition.

In the planning process, we carefully reviewed the re-structuring plans of practically every country that went through the same process. This proved to be a very good learning experience, in regard to the positive and negative consequences.

We've also gained experience from the process of other privatization projects in the Mexican economy.

The learning thus acquired, the valuable intervention of Mexican legislators, the advice of suppliers and clients, and mainly the firm intention of improving the working conditions of the railroaders, have all contributed to the healthy present condition of the program.

As you all may know, this past Monday the opening for bidding for the Chihuahua Pacific Railroad, took place. Next Monday the complementary part will take place and then comes the decision making process. Also, bidding for the concession of the Northeast Railroad was initiated, with plans to reach a decision before the end of the present year.

The participation of very important Companies, mainly railway systems of countries represented here, is due in part to the solid foundation of the program in general, as well as the clear bidding rules that govern the concessions.

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*General Director, NDM

The operation of the railway system through private companies is not new in Mexico. This form of operation of the railway transportation mode was initiated here through concessions to private corporations, which started the construction of the great railroad network we have now.

I would like to emphasize that the "Department of Communications and Transportation" relies upon the solid foundation, clear guidelines and the technical resources, for the process of re-structuring to continue successfully, and thus should allow for the main objectives of the program to take place. The objectives are basically, for the Mexican railroads to be conducive to an efficient, secure and reliable service to the country's economy.

I am sure that because there is a common goal with the AREA in meeting these objectives, that there will be a more active and dynamic participation of the Mexican railroaders in AREA activities.

The infrastructure of railway transportation systems of the countries represented here, symbolizes the link that gives cohesion to this part of the world.

The labor of AREA turns "infrastructure" into ways of "understanding."

Again, I welcome you all, and wish you a very successful 1996 Conference in Mexico, which you may consider your CASA.

Thank you very much and good luck.

OPENING REMARKS AT THE 1996 AREA REGIONAL CONFERENCE MEXICO CITY, MEXICO

By: Edwin L. Harper*

I am very honored to be here on this day of all days—the 175th anniversary of the declaration of the Mexican Republic.

Our three nations—Mexico, Canada and the United States—have different cultures. Yet there are many forces that are driving us together.

The globalization of the economy is one such force. International trade directly or indirectly now accounts for about 30 percent of the revenues on U.S. freight railroads.

Canada and Mexico ranked as two of the United States' three largest trading partners even before the North American Free Trade Agreement. NAFTA will only increase trade among our three nations, especially now that Mexico's economic problems are being addressed.

The AAR also reflects the growing ties among our three nation's. The major freight railroads in both Mexico and Canada are members of the AAR (Association of American Railroads) and represented on our board of directors.

The fact is that the North American rail system is one of the prime assets our three nation's have as they compete in global markets.

All North American railroads operate as a fully integrated system that stretches from the Yukon to the Yucatan. Thousands of freight cars cross international boundaries among our three nations every single day of the year.

The efficiencies of steel wheel on steel rail allow industries in Central Mexico, the U.S. heartland and Canadian prairies to compete successfully in markets all over the world.

While performing this prodigious feat, nothing is more important than safety—and nothing is more important to safe operations than the work so many of you do.

I don't have comparable numbers for Canada and Mexico, so I hope you'll pardon me if I talk just a little about rail safety in the United States.

In terms of train accidents, the past three years have been the safest in railroad history. There has been an impressive 55 percent reduction in train accidents over the past 15 years and a 22 percent improvement since 1990 (Figure 1).

It is also encouraging to note that the number of train accidents that can be attributed to track or signal defects has dropped by more than 13 percent over the past three years—certainly a significant contribution to our improving safety record (Figure 2).

Our freight rail system is the biggest, most productive in the world. Only railroads in Russia and China move volumes of freight approaching those in North America. Our railroads, in fact, move almost three times as much freight as the combined total for all of Europe outside of Russia (Figure 3).

A one-year improvement can be luck. But steady improvement comes only when there is commitment to safety.

The U.S. rail industry has such a commitment, and has back it up with substantial investment.

*President and Chief Executive Officer, AAR

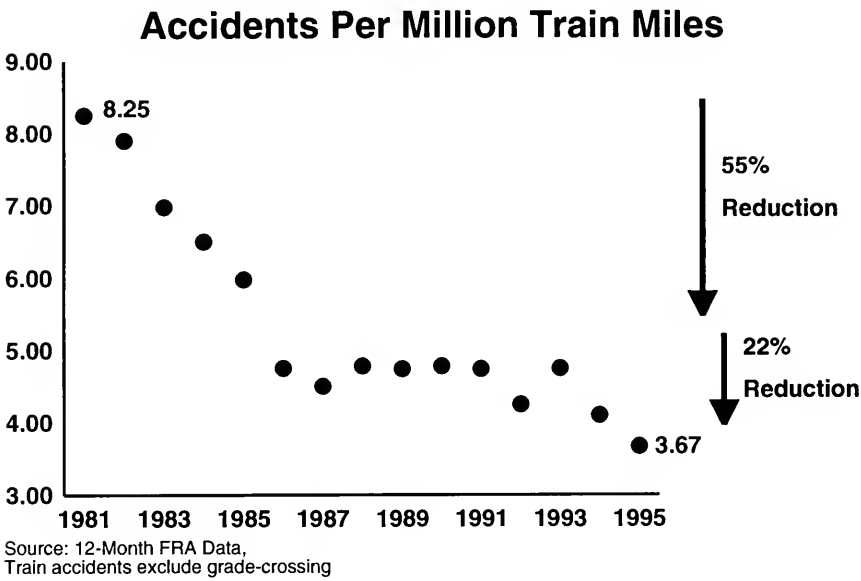

Figure 1. Train Accidents

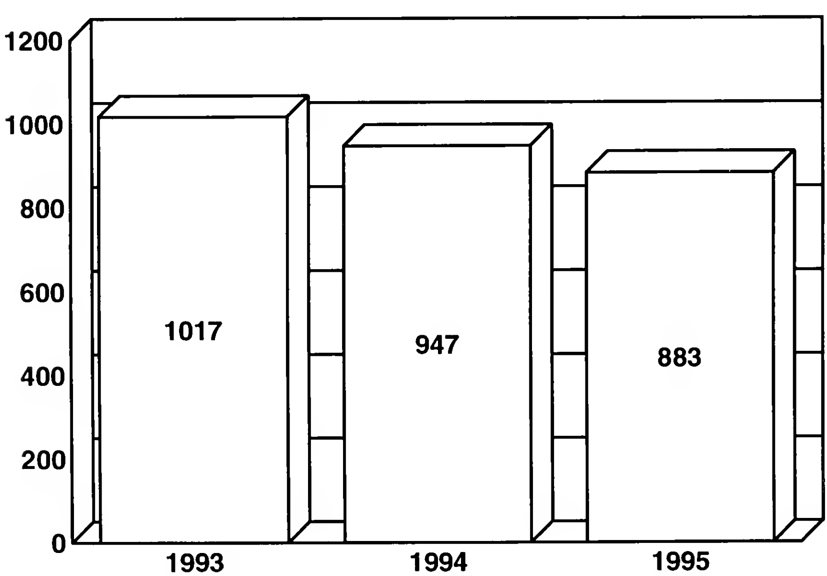

Figure 2. Accidents due to track signal defects are declining.

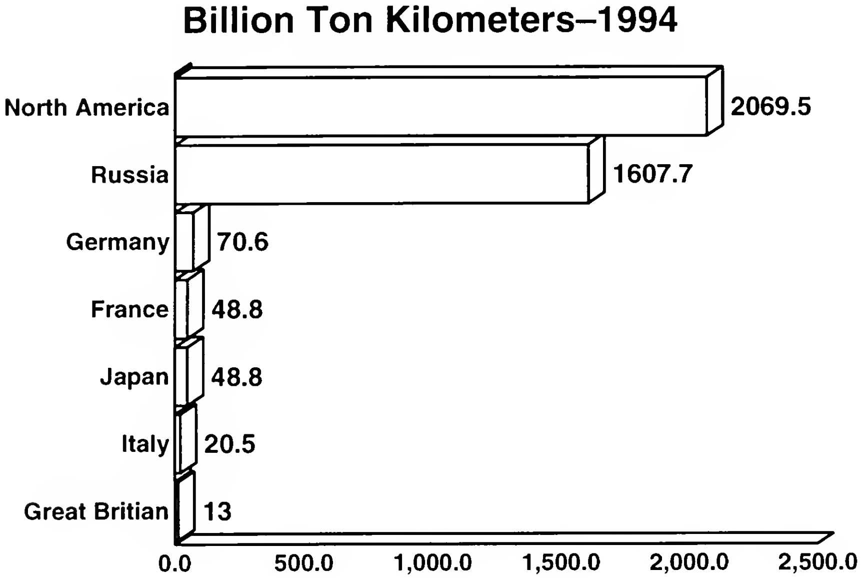

Figure 3. Freight Traffic Around the World.

Just last year alone, U.S. railroads spent almost $7 billion to maintain and improve roadway and structures.

It is true, of course, that safety is not the sole objective of those expenditures. They are also being made in order to maintain and improve efficiency.

And that, precisely, is my point. Safety and efficiency go hand in hand. An efficient railroad will be a safe railroad, because accidents disrupt schedules, damage freight, create service problems and impede operations.

Ours is a great industry. But if it is to do more than just survive . . . if it is to remain relevant . . . if it is to grow, it must undergo constant evolution. And that is what is happening to the railroad industry throughout the world today.

It is what has been happening at the AAR. And it is what has happened with AREA (American Railway Engineering Association).

A few years ago, the AAR Board of Directors decided that the AAR would cease direct support for a number of organizations, including AREA. In effect, they were privatized.

Those for which there was real need would emerge from the crucible of change stronger than ever. And that is the case with AREA. I congratulate you.

I used the term "privatization" to describe the changes at AREA because that is an apt description for what occurred. It is also a trend for railroads that is gaining momentum throughout the world.

Here in Mexico, Ferrocarilles Nacionales de Mexico is now beginning the process of privatization. Canadian National was privatized last year. In the United States, Conrail—which was created from the ashes of several bankrupt predecessors—was privatized in the late 1980s.

Various forms of privatization have occurred in other countries as well—Chile, Argentina, Brazil, England and New Zealand, for example.

The United States for many decades was the only large rail system in the world that was almost entirely privately owned. Yet the U.S. experience shows that privatization by itself is not a panacea. It must be accompanied by a substantial amount of deregulation as well. I repeat, privatization is not enough for a successful railroad system.

U.S. railroads, in fact, stood on the brink of failure during the 1970s, as more than 20 percent of the industry fell into bankruptcy.

Why? Because even though they were privately-owned, they were subject to micro-management by government regulators who could tell them what rates to charge, what services to offer, what routes to operate and even how to operate their equipment.

As a result, railroads were required to subsidize some services and geographic areas. They couldn't drop money-losing services or routes. They couldn't quickly change rates to recover cost increases or take advantage of marketplace opportunities. Nor could they implement their own plans for improving freight car utilization.

They were privately-owned, but regulated for the advantage of their customers and competitors.

The real privatization of U.S. railroads began in 1971. That is when Amtrak was created. Railroads began losing huge sums of money on passenger trains shortly after the end of World War II.

Government regulators required the railroads to continue operations of many of those trains, even as losses mounted to several hundred million dollars annually by 1970.

Amtrak was created as a government-owned corporation to relieve the private freight railroads of that burden. Several years later, Canada took a similar path, creating VIA Rail to do the same thing.

This was the first step in the real privatization of U.S. railroads.

The second step was passage of rail deregulation in 1980. President Jimmy Carter signed the Staggers Rail Act of 1980, which freed railroads from many of controls that had stifled initiative, increased costs and undermined competition.

Although the Staggers Act contained many reforms, I'd like to concentrate here on four that were especially important, four that are critical to the success of a privatized rail system.

The first of these is the ability to freely change rates to recover cost increases and take advantage of market opportunities.

When U.S. railroads were first deregulated, the expectation was that most would increase. That hasn't happened. More rates have gone down that risen.

Why? A couple of reasons. Unrelenting competition from other railroads and from other modes of transportation have prevented most rates from rising. Railroads have also seized the opportunity presented by the market to reduce rates in order to gain new business.

As a result, average rail rates today are less than half what they were in 1980, on an inflation-adjusted basis (Figure 4).

Another key provision of the Staggers Act legalized contracts between railroads and their customers.

Prior to the Staggers Act, all railroad traffic moved under published tariffs. Today, only 12 percent of our traffic moves under tariff. Another 19 percent is totally exempt from regulation (Figure 5).

That leaves almost 70 percent of all rail traffic which now moves under contracts, most of whose terms are private. A typical contract provides customers with rate and service guarantees in return for volume guarantees. This helps ensure a traffic base and aids the railroad in setting schedules and developing long-term strategies.

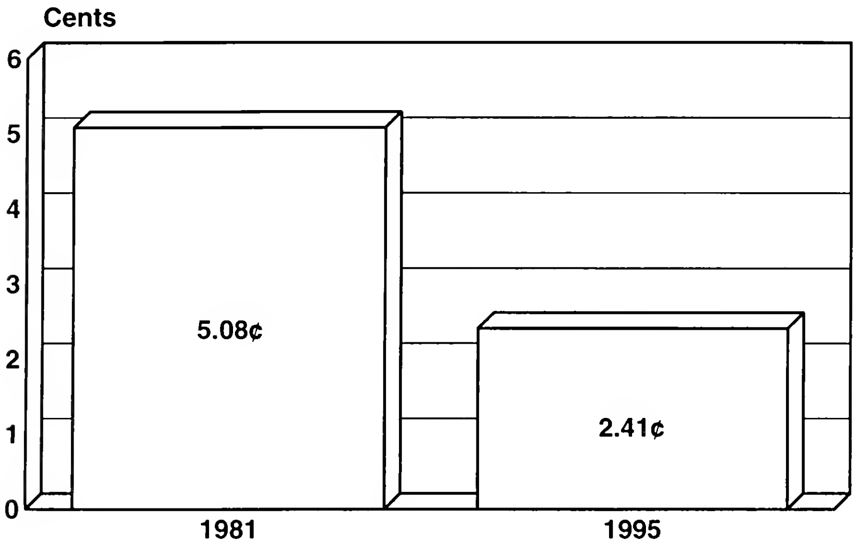

Figure 4. Rail Rates Have Fallen.

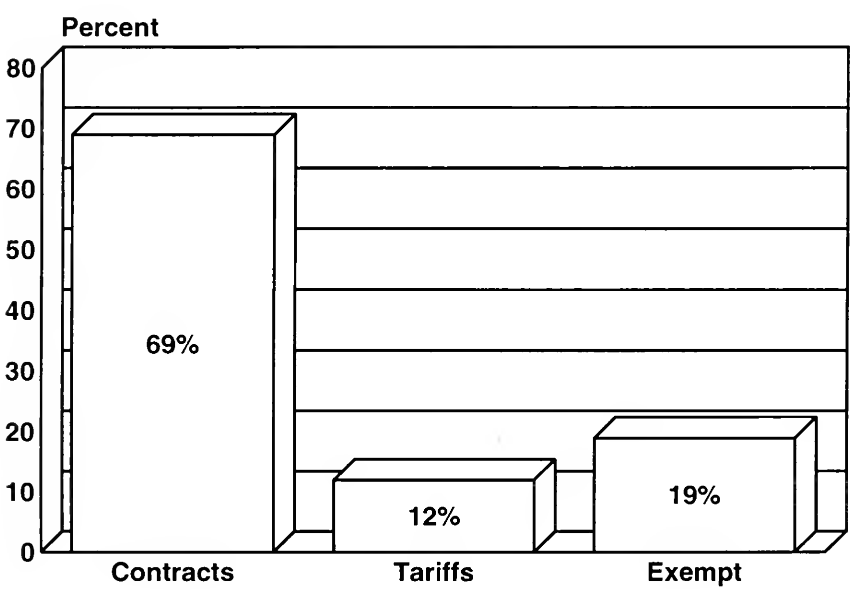

Figure 5. How Freight Moves in the U.S.

Prior to deregulation, railroads were allowed to operate, but not manage their assets. Government bureaucrats could—and frequently did—issue orders telling railroads how to utilize their freight car fleet.

With deregulation, railroads were able to manage those fleets themselves. One result is that the average freight car makes 41 percent more loaded trips per year today than it did in 1980.

More productive use of these assets reduces investment need and costs and increases revenue.

Finally, deregulation spawned two new growth segments in the railroad industry—shortline and regional railroads.

In 1980, Class 1 railroads accounted for 98 percent of all rail revenues. Today, they account for 91 percent.

Under deregulation, Class 1 railroads were freed to sell lines that otherwise would have been abandoned. A new class of entrepreneur has gobbled up many of these lines, seeing an opportunity to provide improved service at lower cost levels.

More than 300 new railroads have been created since 1980, preserving service along more than 30,000 miles of line for thousands of shippers. These feeder lines are responsible for originating almost 20 percent of the 28 million carloads of freight originated on U.S. railroads every year. The freight that the feeders sell then moves onto the mainlines of the Class 1 railroads for delivery to final customers (Figure 6).

As a result of deregulation, U.S. railroads now move substantially more traffic than they did before deregulation. And they accomplished this while substantially reducing route mileage, the number of freight cars and the size of the equipment fleet (Figure 7).

The result has been 15 years of unprecedented productivity improvements. The productivity of our freight cars, track and locomotives have all virtually doubled over the past 15 years while labor productivity has almost tripled (Figure 8).

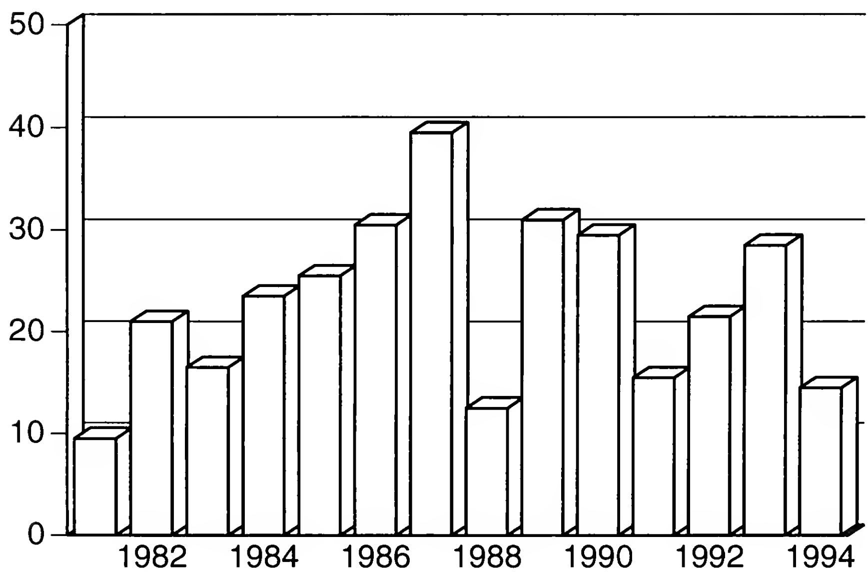

Figure 6. More than 300 New Railroads have been Created.

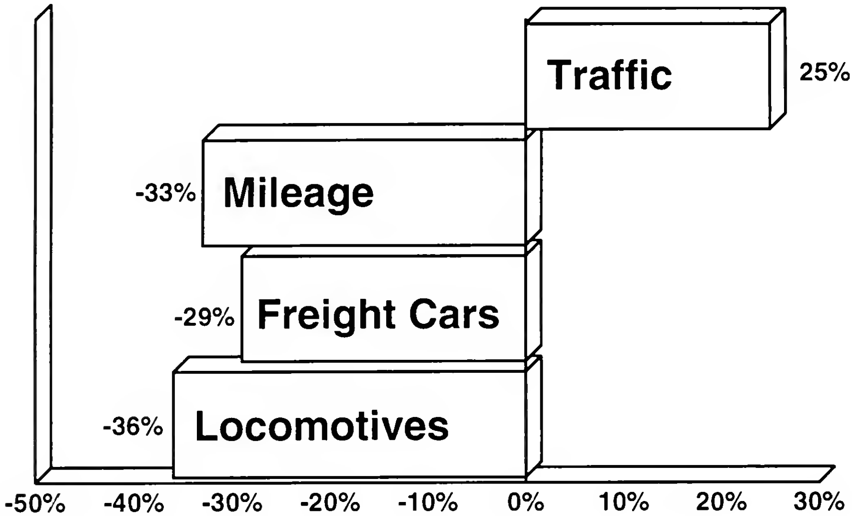

Figure 7. More Traffic, Less Plant and Equipment since 1980.

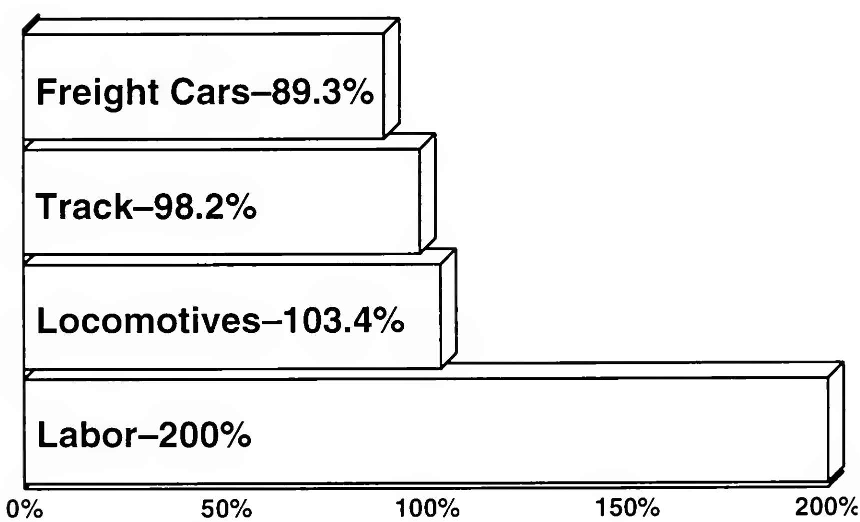

Figure 8. Productivity Gains Since 1980.

All over the world, interest in railroads is growing. Why? Because railroads are safe; efficient; kind to the environment; and save energy. By introducing the disciplines and rewards of the market, privatization can bring railroads closest to realizing their maximum potential in each of these areas.

So this is an industry with a great future. And you are a vital part of that future. No railroad can do any of these things without a well-engineered, well-maintained track structure. It is the most basic element of railroading. It always has been. And it always will be.

MECHANIZED TRACK MAINTENANCE ON THE MEXICAN RAILWAYS

By: Eduardo Ramirez Cato*

It is likely that all of you have seen track maintenance works, but perhaps only a few of you have seen the maintenance work done manually, and less of you have seen the track maintenance work done in Mexico. Starting from this fact, we wish to present to you in a simple manner the evolution of track maintenance work in Mexico, and the immediate perspective for the near future within the frame of aperture to the private investment of National Mexican Railroads.

For a better understanding of why we are where we are, a short summary of the evolution of track maintenance work in Mexico is needed.

The evolution of track machinery in our system has been as follows: in 1944, we started with the track leveling by means of manually operated mechanized 4 and 8 vibrator tamping units. In 1954, the first standard tamping machines with electric vibrators and ladder type rail jacks were used. In 1958, groups were formed with a multitamping machine, an aligning machine with hydraulic jacks and a ballast regulator equipped with pulleys and cables. The graphs of track curves were made manually. In 1960, we acquired multitamping machines with rail raising system, lateral supports and track leveling by means of infra-red beams at 100 feet distance, track aligning machines with anchors and hydraulic ballast regulators. In 1969, we used multitamping machines with arch system, hydraulic aligning machines with clamps, integrated graph system and hydraulic ballast regulator. In 1974, the first multitamping, leveling and aligning machines were used. In 1981, a geometric car was bought. In 1986, switch multitamping, leveling, and aligning machines, with ballast regulators of present technology were bought, and also an undercutting machine. In 1992, we bought four machines for tamping, leveling and aligning and a ballast undercutter. In 1994, two dynamic stabilizers were bought. Presently there are 108 machines in service, that form 33 groups with a total yearly output of 5,765 km. (3,500 miles).

Regarding the maintenance methods evolution, formerly one track maintenance crew took care of 9 miles of track, and consisted of 1 track foreman, 1 track keeper and 5 repair men, attending only spot maintenance, while the ambulatory or regular track crews were distributed one per district for intensive track care, consisting of 1 track foreman and 20 repair men. All work was carried out manually.

Back in 1944, the American Mission created temporary crews formed by 1 track foreman and 25 repair men, which later in 1947 became system crews to attend the widening of narrow tracks from 0.915 to 1.435 m. and track rehabilitation with maximal output of 500m. These crews consisted of 1 general foreman, 5 track foremen and 125 repair men, using 110 to 112 lb/yd, 39' rail over wooden ties.

In 1964, the first semi-mechanized rail change was made by welding 115 lb/yd rail in sections of 4 rails, with thermite welding, over concrete biblock ties.

In 1970, the first mechanized change of rail was made using 115 lb/yd plant welded rail, over concrete ties with double elastic fastening. With trained personnel this task resulted in faster advance and better quality. This work was focused on the priority basic network and were the first steps to a complete mechanized track maintenance.

Starting in 1992, a voluntary retirement plan was initiated, which was accepted by a high percentage of personnel involved with manual track rehabilitation and maintenance. To make up for the scarcity of labor and to make more efficient use of the present work force, a decision was made to

*Technical Coordinator—Track and Structures, FNM

apply complete mechanized systems. This has been achieved with the machinery groups presently in service, and with partaking of private investors which had never participated before in this kind of work.

A fundamental part for this decision was the creation of High Systematization Tandem groups called TAS for their spanish name. The basic concept for forming these leveling groups arose from the need to give support to the regions forming the National Railroads with groups of multitamping machines, of one single brand, and high output ballast regulators. Since the allowed time of track occupancy for maintenance is very short, it was established that in a 50 km track length, three leveling groups must work simultaneously in TAS.

The following advantages were obtained from these tandems:

- Shortest track occupation time,
- Less train delay,
- Increased machinery output,
- Elimination of slow orders in a quicker manner,
- Increased track personnel efficiency,
- Lower machinery maintenance costs,
- Lesser inactivity periods, and
- Increased safety and train speed

In order that the jobs with the tandem groups have an appropriate guarantee in quality and durability, track preparation is required to obtain the best results. Such preparations include:

- Damaged ties replacement,
- Inspection, lubrication and replacement of rail fastening devices,
- Gage correction,
- Correction of tie's spacing,
- Replacement of damaged rails,
- Refurbishing of level crossings,
- Cleaning and construction of ditches and counter ditches,
- Culverts cleaning, and
- Discharge and distribution of ballast.

Once the preparation work is ready, TAS groups carry out the track geometry restitution work with the following main activities:

- Raising and tamping,
- Leveling and aligning,
- Ballast regulation,
- Shaping and ballast combing, and
- Track stabilization, in some cases, with mechanical means.

The benefits thus far obtained include among other things, an increase in time between the normal track maintenance cycles.

The production output with TAS has an average of 6 km per day, with a maximum of 12 km in a daily work yield, depending on the windows on the track and the machinery type. These figures do not include machinery transportation time to the work site. This contrasts with machinery groups working separately with their yield rarely going over 1 km per day, and which require continuous care of the mechanic personnel, coupled with a constant delay of trains due to the movement of equipment to the nearest siding.

There are presently in the National Railroad System, five TAS groups located at:

TAS 1. In the North East Railroad, with Plasser equipment.

TAS 2. In the North Pacific Railroad, with Plasser equipment.

TAS 3. In the South East Railroad, with Jackson equipment.

TAS 4. In the North Pacific Railroad, with Matisa equipment.

TAS 5. In the North Pacific Railroad, with Tamper equipment.

The last data about the yield of these groups is as follows:

TAS 1. 4,100 km in 34 months,

TAS 2. 2,100 km in 22 months,

TAS 3. 600 km in 11 months,

TAS 4. 49 km in 2 months,

TAS 5. 15 km in 1 month (test period).

The total output of all groups together reaches 6,864 km.

With the reduction in personnel, spot maintenance was re-structured by forming high efficiency mechanized groups in replacement of the former crews. The territory assigned to each group is 120 km. in length. Each group has a Hi-rail truck equipped with basic tools to carry out minor maintenance work. This work mode was initiated in 1991 and presently 8,000 km. have been covered.

Track Maintenance Parameters

Since the beginning of the 80's, the National Railroads of Mexico introduced a car for recording track geometry defects with the most modern technology recently updated. Based on these reports, it has been possible to do track maintenance work in a more efficient manner. The geometric parameters assigned for recording are constantly reviewed, in connection with the quality standards levels established for the National Railroad System.

Recently, in connection with the creation of Regional Railroads, new technical parameters were established to assure operational safety. In this sense, a general classification was made of all existing tracks, in such a way as to establish equivalent competitive parameters of all the railroads as a function of the importance of each track.

The classification of the tracks was made taking into account the gross annual tonnage transported over each line, and the speed developed in the same track, covering the five last years. The higher transported tonnage and speed corresponds to a higher line importance, and therefore require more attention to the corresponding geometric parameters and physical conditions. Based on these criteria, 6 track importance classes were established, assigning class 1 to the main tracks with higher tonnage and class 6 to branch tracks.

The class index was calculated with the following empirical formula:

$$I = T \times 1.01^{V}$$

where:

I = Track importance index

T = Annual gross tonnage in the track section, in million metric tons.

V = Maximal speed of the fastest trains, in km/h.

From the results obtained with the formula, the class ranges were defined as depicted in Table 1.

The general spectrum of this classification is shown in the following graph (see Figure 1).

Track classification is not meant to be static. It must be updated when the operating conditions change, and consequently geometric and physical conditions must correspondingly change.

Once this classification was established and corresponding tolerance limits for the main geometric and physical parameters were assigned, these became an equitable base upon which the corresponding railroads must apply their track resources as a direct consequence of the importance index. Therefore, the higher the importance index, the higher the track resources needed for its adequate maintenance.

Table 1.

Track Class	Importance range
1	49.9 or higher
2	30.7 to 49.9
3	15.3 to 30.6
4	7.7 to 15.2
5	2.7 to 7.6
6	0.0 to 2.6

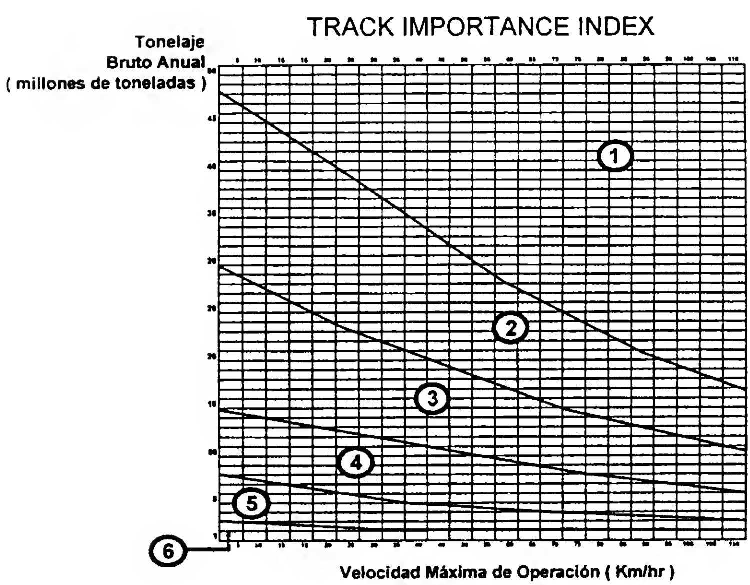

Figure 1.

The limits were established taking as bases the different studies made by AAR and the AREA commendations, also based on the internal bulletins of some North American Railroads and FRA regulations, and finally according to the own experience of National Mexican Railroads regarding the operation, the track maintenance personnel, available resources and the geographical application zone.

The track condition is a result of the physical status of its elements and their geometric conditions. If there is a defect in any track element, it causes a failure in the track geometry, the opposite is also true, one condition depends on the other, therefore, attention is required for both concepts in an integral manner.

The limits regarding the geometric track conditions according to its class are shown in Table 2.

Some of the limits regarding the physical status of the constituent track elements, according to the category are shown in Table 3.

If the track conditions do not fulfill the established tolerances, that track must be classified in the next lower level where they are all fulfilled.

Table 2.

Parameter		**1**	**2**	**3**	**4**	**5**	**6**
Alignment failure	mm	11	12	17	21	27	35
Leveling failure	mm	14	16	16	19	19	21
Short warping	mm	10	14	20	25	30	35
Long warping	mm	32	44	47	51	55	55
Over elevation difference	mm	14	18	22	26	30	34
Open standard gage difference	mm	19	25	25	25	25	32
Closed standard gage difference	mm	6	6	6	6	6	6
Minimum crown width in road bed	mm	6.6	6.0	6.0	5.5	5.0	4.0
Minimum ballast thickness	cm	30	26	22	18	14	10
Minimum ballast shoulder width	cm	30	25	20	15	10	5
Maximum positive gap in Welded joints	mm	1.0	1.0	1.0	1.0	1.6	1.6
Maximum negative gap in Welded joints	mm	0.0	0.0	0.0	1.0	1.0	1.5
Deviation of the rail head internal face	mm	0.5	1.0	1.5	1.5	1.6	1.6

Table 3.

Parameter	**Unit**	**1**	**2**	**3**	**4**	**5**	**6**
Maximum number of defective ties per km	piece	50	60	80	120	160	200
Maximum number of defective ties per group	piece	0	2	4	4	5	5
Minimum rail gauge	lb/yd	115	112	110	100	90	80
Maximum rail head wear (vertical-horizontal)	mm	10	10	12	12	14	14
Maximum flattening in joints with shoes	mm	0	5	5	6	6	6
Maximum number of internal rail defects in 10 km	Nr.	5	10	15	20	25	30
Maximum number of missing bolts in each end of the shoe	piece	0	0	0	0	1	1
Maximum number of loose nails for 24	piece	0	2	4	4	4	6

As an assistance to the Division Engineers, in order that they may periodically prepare a diagnostic of the track condition, and thus schedule the maintenance or corrective work as needed, a qualification sequence has been prepared for each of the track parameters.

The following concepts are qualified by their weighed overall importance within the track structure:

Rails: The rails qualification is weighted as 30% of the global track qualification.

- Gage
- % of wear
- Plastic deformation
- Deformation at joints
- Corrugation
- Sliding
- Internal defects
- Visible fatigue

Ties: The ties qualification is weighted as 20% of the global track qualification.

- Groups with defects
- % of defective ties per kilometer

Fastenings: The fastenings qualification is weighted as 11% of the global track qualification.

- Sets out of adjustment
- Sets in bad physical condition

Ballast: The ballast qualification is weighted as 11% of the global track qualification.

- Ballast thickness
- Ballast shoulder width
- Ballast contamination

Roadbed: The Roadbed qualification is weighted as 15% of the global track qualification.

- Roadbed section
- Roadbed slopes or cuts
- Chronic road bed settlement
- Water saturation in road bed
- Ditches and counter ditches

Geometric track conditions: The geometric track conditions qualification is weighted as 13% of the global track qualification.

- Mis-alignment
- Steps in leveling
- Standard gage
- Over elevation

With all the above, a qualification from 1 to 10 is obtained. Then, the Division Engineer has to arrange some corrective action to guarantee the operational safety and from this, the schedule for utilization of the track machinery in those areas deemed more critical.

This way, it is foreseen that the Transportation and Communications Ministry will establish some criteria for supervision of the track maintenance work within the regulatory frame to be made for the concessioned railroads, and thus fulfill one of the objectives in the privatization process, which is to have a safe, efficient and competitive railroad transportation system.

Conclusions

The evolution is the mechanized track maintenance in Mexico has been looking for an adjustment within the budgetary limits towards the most modern technologies.

The National Railroads of Mexico has taken, in its moment, the decision of modernizing its tracks, together with the logical process of maintaining them. With this perspective, the Mexican Government intends to continue the present tendency, and therefore it is one of the bases for the aperture to the private capital investment under way.

When incorporating the TAS groups for mechanized maintenance, besides replacing FNM's labor force which is under re-dimensioning, it has been proved it is the basis which assures a good track quality, in an appropriate cost-benefit balance.

The track maintenance parameters are the tools which warrant a track quality standard under equitable operating conditions.

The methodology for track qualification, is a dependable base to establish application criteria for human, material and economic resources and for establishing the annual maintenance work schedules.

INITIAL RESULTS OF FAST/HAL PHASE III TESTING

By: David M. Read* and Semih Kalay**

Abstract

Initial results from the operation of 315,000-lb cars equipped with improved suspension trucks at the Facility for Accelerated Service Testing (FAST) suggests there are substantial financial benefits through savings in fuel usage, wheel/rail wear and other reductions in track damage. Data collected during Phase III of the FAST program has enabled engineers from the Association of American Railroads (AAR) to determine, for the first time in recent railway research, the overall savings in track and operating costs due to improved truck suspension systems. Prior to this experiment, benefits were estimated through models and expert opinion.

FAST Phase III operations with improved suspension trucks began in November 1995 and have produced 145 million gross tons (MGT) of traffic to date. The test trucks were designed primarily to reduce lateral wheel/rail forces through improved curving performance. A cursory look into fuel consumption at FAST indicates the improved trucks provide up to a 27 percent reduction in fuel usage as compared to previous operations with standard trucks.

Initial results also indicate that lateral forces measured on medium degree curves (5-degree to 6-degree) at FAST are approximately 50 percent of those measured previously with standard suspension trucks. Rail wear, especially gage face curve wear, has also been reduced at least 50 percent for both standard and premium rails. The improved curving performance appears to have also reduced rail corrugation development. Rail corrugations which were visible at 60 MGT of standard truck operation, are not evident after 140 MGT of improved suspension truck operation. Gage widening of softwood ties on curves has also been reduced by slightly over 50 percent due to the improved curving characteristics.

Background

Phases I and II of the Heavy Axle Load (HAL) program at FAST, investigated track performance under 315,000-lb vehicles (39-ton axle loads) equipped with standard design three-piece freight car trucks. Phase I, in which 160 MGT of traffic was accumulated over the 2.7-mile High Tonnage Loop (HTL), quantified the effects of 39-ton axle loads on standard track components and provided a comparison with track performance previously measured under 263,000-lb vehicles (33-ton axle loads). Results of Phase I indicated that 39-ton axle loads could operate on a standard track structure, but with increased track maintenance costs of approximately 30 percent.

During Phase II, premium track components were introduced to determine if an improved track structure would reduce the maintenance penalty measured in Phase I. After 300 MGT of Phase II testing, results indicated that premium materials, especially head-hardened rail, concrete ties with dual durometer or "sandwich" tie pads, high integrity frog castings, premium thermite welds, and hardwood ties with elastic fasteners would improve the safety, efficiency and economics of 39-ton axle load operation.[1,2]

As Phase II was in progress, planning for a third phase of the program was initiated to determine the benefits of operating 39-ton axle loads with improved suspension trucks. Ten potential truck designs were evaluated analytically using the NUCARS model and then field tested to Association of American Railroads (AAR) Chapter XI New Freight Car Service Worthiness criteria. From this group, three designs were selected in early 1995 for Phase III operations, the FAST train was re-equipped during the summer and operations began in November 1995. Current Phase III tonnage is 125 MGT with 160 MGT expected by the end of 1996. A brief description of the improved suspension trucks is as follows:

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- Thirty-seven cars equipped with Standard Car Truck's heavy duty S-2-HD frame brace design with variable column damping, D5 secondary suspension with double side coils and elastomeric shear pads between the bearing adaptor and side frame for primary suspension.
- Thirty-six cars equipped with Buckeye Steel Casting's XC-R VII design with constant column damping, increased friction wedge surface area, D5 secondary suspension with hydraulic snubbers and primary suspension shear pads.
- Four cars equipped with the American Steel Foundries' (ASF) AR-1 passive radial steering design with constant column damping, D5 secondary suspension with hydraulic snubbers and primary suspension shear pads.

In general, all three trucks are designed for improved curving response with enhanced wheel set steering capability and resistance to truck warp. Steering is provided by the longitudinal and lateral stiffness of the shear pads and the steering arms of the AR-1 truck. Truck warp is controlled by the cross bracing of the S-2-HD trucks and the increased friction wedge surface of the XC-R VII truck. In addition to improved curving, the elastomeric characteristics of the primary suspension pads have the potential of attenuating high frequency vertical forces. Other than the hydraulic dampers on the Buckeye and ASF trucks, secondary suspension designs are not substantially different from the standard design trucks. Low frequency vertical forces, therefore, are not expected to be reduced, particularly at non-resonant vehicle speeds.

Initial Results

Dynamic rail force data was collected in the 6-degree curve of HTL Section 25 after accumulation of 110 MGT to quantify curving performance of the improved trucks. Similar data had been collected from the standard truck consist at the same location and under comparable wheel/rail profile and lubrication conditions a year earlier for comparison purposes. Figure 1 compares the median (50th percentile) and 90th percentile lead axle lateral forces measured under the standard and improved trucks. The force values in Figure 1 were measured on the high rail with the gage face of the high rail lubricated and the train operating at 40 mph. The data shows the lead axles of the improved trucks are generating lateral forces at the measurement site approximately 50 percent lower than the standard trucks.

Wheel set angle-of-attack was also measured at the same 6-degree curve location used to measure lateral forces. The lead axle of a standard freight car truck will generally assume an angle-of-attack relative to the curve radius in milliradians roughly equal to the degree of curvature while the trail axle tends to remain radial to the curve with a very small angle-of-attack. In Figure 2, a comparison of 50th percentile and 90th percentile lead axle angle-of-attack data is shown. The data indicates a 50 percent reduction in lead axle angle-of-attack for the improved trucks.

The improved curving performance has resulted in a 50 percent to 60 percent reduction in rail wear. Rail wear was measured on a variety of rail types, including head-hardened and standard rail, in the 5-degree curve of Section 07 during the last 85 MGT operation of the standard trucks. Identical rail types were installed in the same curve and wear measured during the initial 85 MGT of improved operation . The average total wear measured under both truck types is shown in Figure 3 and a comparison of standard rail profiles from Section 07 measured after 85 MGT is included as Figure 4. The percent reduction in gage face wear agrees closely with the percent reduction in curving forces.

In addition to reduced rail wear, improved curving performance has deterred development of rail corrugations, reduced fuel consumption and reduced gage widening of softwood ties. Measurable corrugations were present in the standard rail in Section 07 beginning at 60 MGT during the standard truck operation. However, no corrugations were apparent after 100 MGT of improved truck operation in the same curve. It is conjectured that improved truck steering has resulted in lower wheel/rail longitudinal forces and contact stresses and, therefore, reduced the tendency toward corrugation formation.

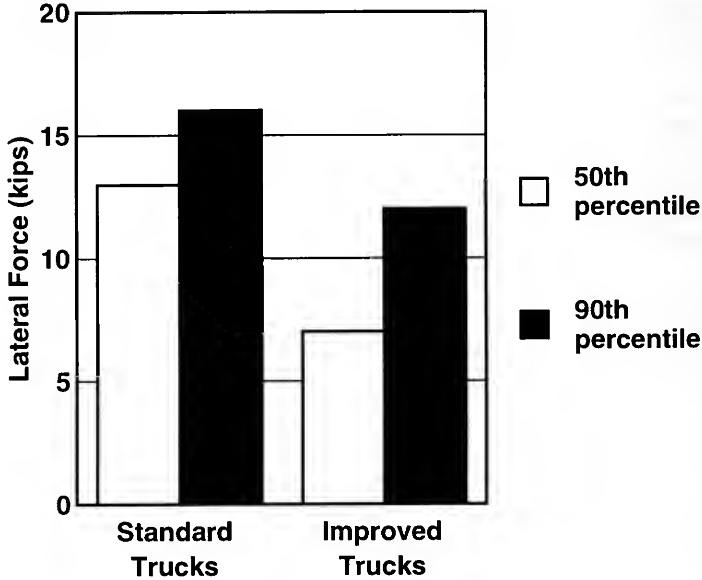

Figure 1. Comparison of 50th and 90th Percentile Lead Axle Lateral Force Values

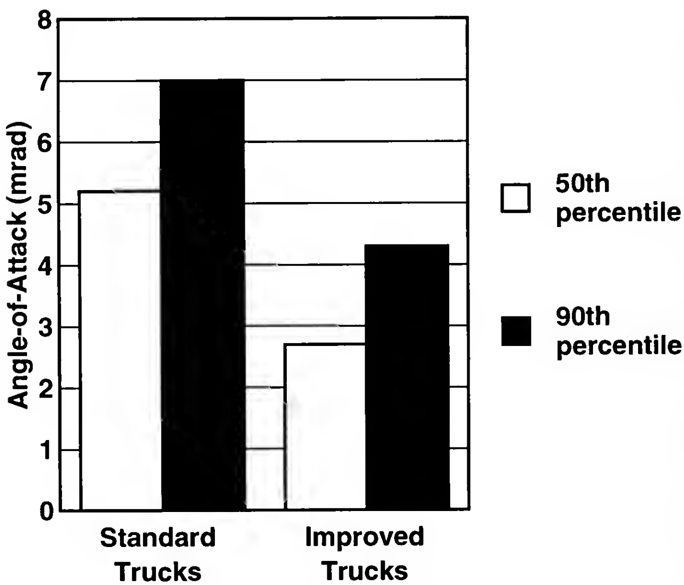

Figure 2. Comparison of 50th and 90th Percentile Lead Axle Angle-of-Attack Values

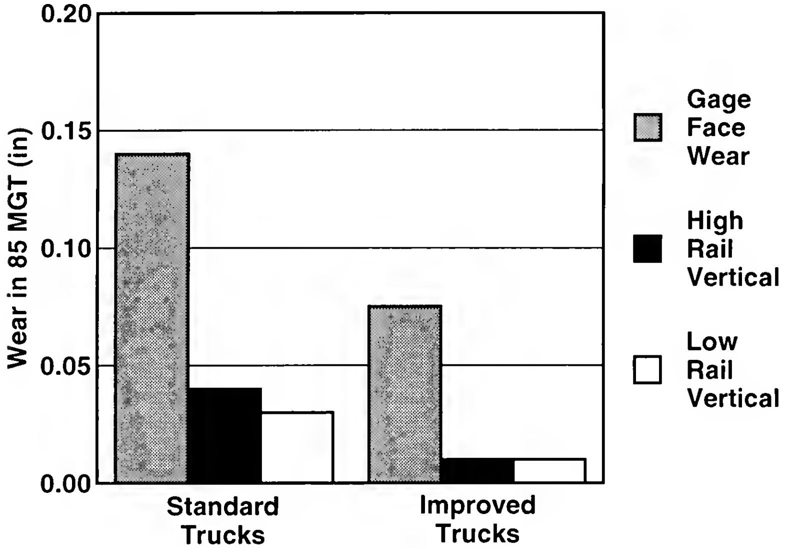

Figure 3. Comparison of Average Rail Wear

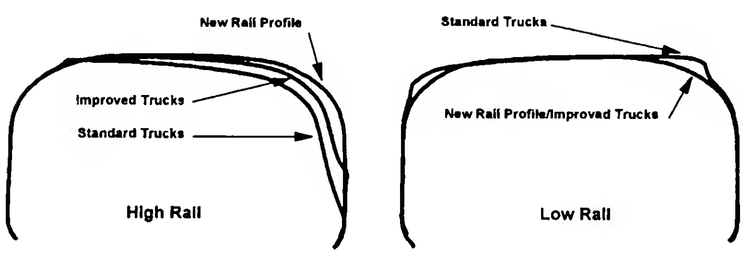

Figure 4. Comparison of Standard Rail Profiles After 85 MGT

Fuel consumption dropped approximately 27 percent from an average of 3,750 gallons/MGT during the final 32 MGT of Phase II to an average of 2,740 gallons/MGT during the initial 100 MGT of Phase III. The locomotives were identical for both Phases and the consist weight varied between 11,500 tons and 12,200 tons. A back-to-back comparison of gage widening on southern yellow pine ties located in Section 25 showed a substantial reduction of about 60 percent with the improved trucks.

Other preliminary results suggest that the primary suspension pads are effective at attenuating high frequency vertical forces at thermite welds and rail joints. After 60 MGT of standard truck operation, the average batter measured at thermite welds located in a 5-degree curve was 0.040 inches as compared to 0.012 inches measured after 60 MGT of improved truck operation over similar welds installed at the start of Phase III on the same curve. The average rail end batter in mechanical joints installed in a tangent section and a 5-degree curve was also reduced by about 70 percent with the introduction of the improved trucks.

It should be noted that the savings in fuel and track damage costs achieved at FAST may translate into less savings (although still very significant) in revenue service. This is due largely to the fact that the FAST experiments are well controlled and the track and equipment maintenance procedures are unique to the FAST operating environment.

Summary and Future

Data collected during the first 100 MGT of Phase III operations indicate that the improved suspension trucks are:

- Generating lower lateral forces on medium degree curves.
- Providing significant reductions in rail wear, fuel consumption, gage widening and rail corrugation development.
- Producing less batter at rail joints and thermite welds.

However, the trucks are all new and operating at near optimum performance. More mileage is required to determine if the initial results will diminish as the trucks wear. Phase III is currently planned to continue through 1998 and generate an additional 300 MGT of traffic. A preliminary economic analysis is currently underway to determine whether the cost savings can justify the added investment due to improved trucks as well as to quantify the effects on overall HAL operations.

References

1. Read, D. M. and S. Kalay, "Results of Phase II Heavy Axle Load Tests at FAST," American Railway Engineering Association Bulletin 757, Volume 97, October 1996.
2. Hargrove, M., T. Guins, D. Otter, S. Clark, and C. Martland. "Economics of Increased Axle Loads: FAST/HAL Phase II Results," A World of Change, 1st Annual AAR Research Review Proceedings, Volume 1, November 6–9, 1995.
3. Klauser, P., C. Urban, and R. Florom. "On-Track Test Results for the Heavy Axle Load Alternative Suspension Project," AAR Report No. R-896, October 1996.

ERRATA FOR PAPER BY G.J. MOYAR AND D. H. STONE ENTITLED "HIGH ADHESION LOCOMOTIVE THERMAL-MECHANICAL RAIL SURFACE LOADING," AREA BULLETIN NO. 756, MAY 1996, PROCEEDINGS VOLUME 97 (1996), PP. 407–412

Final format changes in the printing caused some errors in Equation (2) on page 410 and Table 1 on page 411 that, unfortunately, were not corrected in final proof reading. Important corrections are provided below.

The correct form of Equation (2) is:

$$TMSI = \frac{p}{k}\mu L^{1/2}[(1+r)^{1/2} - 1] \tag{2}$$

The correct form of Table 1 is:

Table 1. Comparison of AC to DC Tread TM Starting Loading Severity for 154 kN (34,614 lbs.) on 1.016 m (40") wheel @ 3.1 m/s (7 mph) speed

	AC	DC	% Change from AC
Adhesion %	45	35	–22
Slip %	10	5	–50
Hertz Pressure (MPa)	1142	1142	0
Flash Temp. Rise (°C)	372	146	–61
TMSI	3.6	1.2	–68
Max. Eff. Stress (MPa)	1190	798	–33
Max. Sxx Range (MPa)	2436	1848	–24
Max. Shear Stress (MPa)	648	446	–31
Max. Resid. Stress (MPa)	505	70	–86
Plast. Shear Strain (%)	0.911	0.311	–66

Finally, the label is missing for the vertical axis in Figure 1. It should be "Temperature, °C."

ERRATA FOR
AREA COMMITTEE 1 REPORT ON "MILL ABRASION TEST STUDY" PUBLISHED IN AREA BULLETIN NO. 756, MAY 1996, VOLUME 97, PP. 419–423

The subject of the Committee's informational report, "Mill Abrasion Test Study," was inadvertently omitted in the Contents of the May Bulletin.

The study was conducted by Subcommittee 2, and not by an individual as shown. Mr. J.K. Lynch, Chairman of Subcommittee 2, is a former employee of Vulcan Materials, and is now a private consultant.

EFFECT OF RAIL BASE MISMATCH ON IN TRACK ELECTRIC FLASH BUTT WELD SLOW BEND PROPERTIES

By: C. P. Lonsdale*, S. E. Markis and A. E. Shaw*****

Abstract

Nine welds were produced using a Holland mobile in track electric flash butt welder in order to determine the effect, if any, of the amount of rail base mismatch on slow bend properties. The first three welds were joined with no base mismatch. The second three welds were joined with a nominal base mismatch of 5⁄16 inches. Because the physical limit of the welding head is 5⁄16 inches mismatch, the amount of mismatch was slightly less. The last group of welds had 1⁄8 inches rail base mismatch.

All nine welds were then slow bend tested. Five of the welds reached two inches of deflection and testing was stopped prior to failure. Four of the welds fractured. All welds met the specified values of modulus of rupture and deflection for A.R.E.A. slow bend testing. Welds with no mismatch had the best average load, modulus of rupture and deflection values followed by welds with 1/8 inches mismatch. Welds with 5⁄16 inches mismatch had the worst average values. There was substantial overlap in the 95% confidence intervals for load, modulus of rupture and deflection values among the three weld groups. Three of the fractured welds had obvious fracture defects and one of these also exhibited "flat spots" on its fracture surface. One weld showed only "flat spots". The welds with obvious defects had worse average values for properties than the weld with only "flat spots".

Introduction

The April, 1996, meeting of the American Railway Engineering Association Committee 4 in Roanoke, VA, brought up an issue which has long been discussed in rail welding circles. The question of how much mismatch between rail sections should be allowed, and where such mismatch should be located (base or head) is an important concept. During flash butt welding with mobile in track machines a wide variety of rail sections can be encountered including severely curve worn and older, more uniformly worn rails. As railroads continue to attempt to extend the life of one of their major assets, rail, the problem of repair and maintenance welding of different sections with mobile flash butt welders will increase.

During the Subcommittee 1 (Welding of Rail) meeting, Chairman Phil Lewis suggested that a working group investigate the issue of rail mismatch and perhaps work towards a specification of some kind. Several members volunteered to join the effort and a meeting was held at Pohl Corporation in Reading, PA, on May 6, 1996, to begin work. Meeting attendees included Mr. Phil Lewis of Lewis Rail Service/Holland Company, Mr. Al Shaw of Amtrak (retired), Mr. Charles Deal of Pohl Corporation, Mr. Richard Morris of Holland Company, Mr. S. E. Markis of Conrail and Mr. C. P. Lonsdale of Conrail.

Background

The group first outlined the different types of rail mismatch situations that are encountered when using a mobile in track electric flash butt welder. These cases are listed below:

- Insulated joints—new and relay.
- Welding at start/end of rail laying job.

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- Relay rail—CWR program.
- Plug rail/maintenance welding.
- Road crossing renewal.
- Turnouts—new to body of track.
- High/low side of curves.
- Fill in rails for out of face cropping.

Given that the maximum tensile bending stresses occur at the outer fibers of the rail base, ideal weld strength occurs when the webs and bases form a perfect match. When welding together like sections of rail with varying amounts of vertical wear and dissimilar sections of rail, speed and practicality often dictate that it is more desirable to line up the running surface with mismatch in the base and web. This, of course, minimizes batter at the weld account of imperfect grinding and reduces the amount of grinding needed to be performed by track workers. When welding dissimilar rails together with mismatched bases, one should make sure the weld is in a crib. Placing a weld with a mismatched base upon a tie plate may create high stress concentrations account of the offset and the likelihood of little or no grinding on the underside of the base. This sometimes may be difficult to do without respacing ties, which is time consuming and expensive.

Current FRA safety standards provide a table of the mismatch allowed at rail joints, but this applies more generally to bolted joints (Ref. 1). Mr. Deal provided a grinding specification used by Teleweld for grinding rail joints many years ago (Ref. 2). This specification, used when bases are matched and mismatch is accounted for at the rail running surface, required measuring the difference in heights of the two rails in thousandths. This reading is then divided by 0.007 to determine in inches the lengths of the ramps to be ground which will provide a smooth transition between heads of rails with different heights.

An example using this formula for a new rail welded to one with 1/8 inches vertical wear with bases matched, shows that eighteen inches of grinding is required to have a smooth running surface transition between the two rails. To see if this old practice is still valid today with our heavy cars, we asked the A.A.R. to perform computer modeling on a wheel passing over this ground joint in both directions at 80 miles per hour. This was done using NUCARS 2.1 using their generic covered hopper car with a 33 kip wheel load. The results showed a maximum wheel impact of approximately 50 kips for the car moving "up" the ramp and approximately 45 kips for the car moving "down". Since Conrail's wheel impact detector is set to alarm at 100 kips, the old formula still provides valid information. Figures 1 and 2 from the NUCARS simulation show the vertical wheel loads as the freight car passed over the joint. Additionally, the wheels of the car did not leave the rails in either direction according to the computer model.

Holland Company produced several mismatched electric flash butt welds in 1990 which had the offset accounted for at the head (Ref. 3). These welds, produced in track with running surface offsets ranging from $\frac{1}{16}$ to $\frac{1}{4}$ inches, accumulated 80 MGT of rail traffic at the A.A.R. FAST track without failure by the time the report was written. The surface ramp grinding formula used to blend the offset at the head is based on the weld crown/vertical offset allowed by A.R.E.A. (Ref. 4). Three times the offset (inches) divided by 0.060 gives the length of ramp grinding needed at the weld. The value for $\frac{1}{8}$ inches head mismatch would be 6.25 feet. This is substantially greater than the ramp needed using the old Teleweld formula, although the wheel impact force is likely less.

Rail welds with mismatch are usually made with the offset taken at the base for ride quality and in order to save grinding time and money. As a result, it was felt that a test plan should be established to quantitatively link mechanical properties with the amount of rail weld base mismatch. Although Holland Company stated that the practical limit of rail mismatch for their mobile in track welder is $\frac{5}{16}$ inches due to the welding head, establishing quantitative relationships would allow for better welds. The properties of each mismatched weld could then be compared to the existing A.R.E.A.

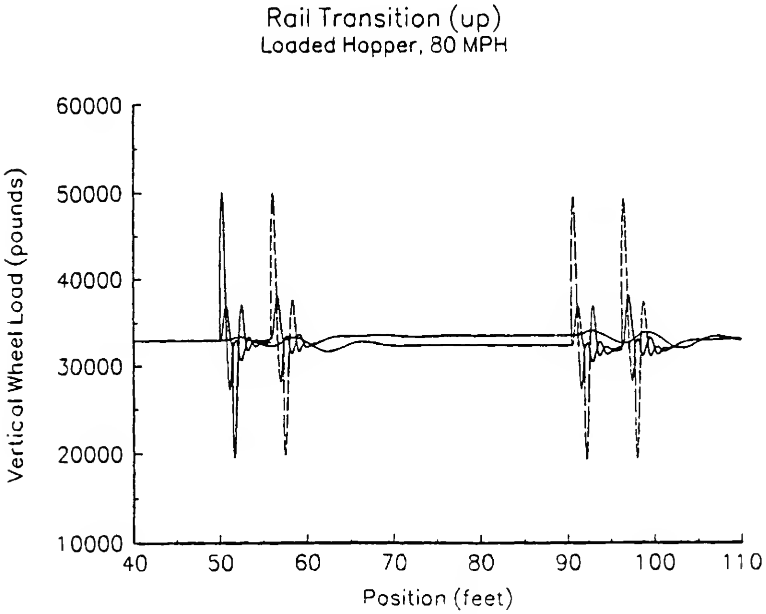

Figure 1. Wheel loads of car moving up ramp.

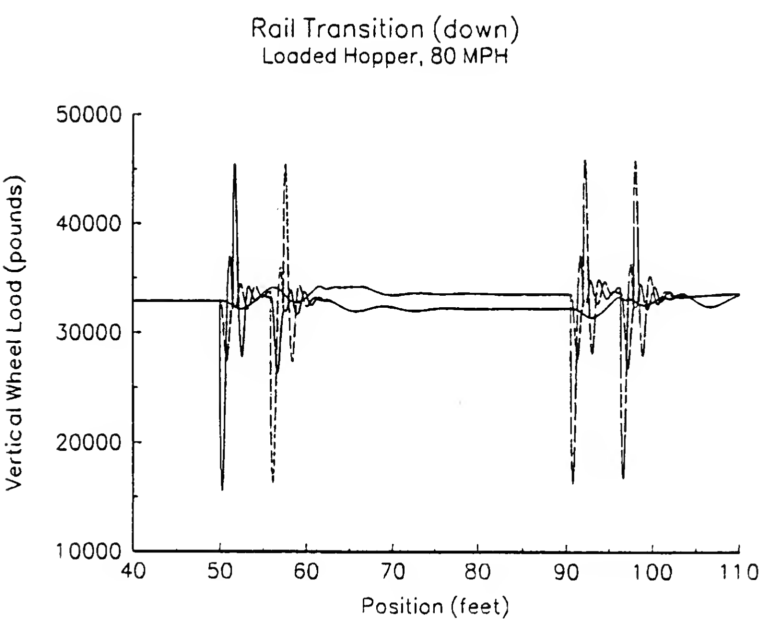

Figure 2. Wheel loads of car moving down ramp.

specifications for electric flash butt welding of rail (Ref. 5). Comparisons between welds of varying base mismatch amounts as well as to mobile flash butt welds with no mismatch could also be made.

Scope of Evaluation

In order to determine the effect of rail base mismatch on mechanical properties, nine welds were to be made using the Holland mobile in track electric flash butt welder. Three welds were to be made with no base mismatch, three with 1/8 inches base mismatch and three with 1/4 inches mismatch. Each weld would then be transported to Lehigh University in Bethlehem, PA, for slow bend testing. Load and deflection values would be collected and the data analyzed. The welds were to be made on the last day the Holland welder was here on Conrail and Conrail would provide rail, crews and transportation for the finished welds. Conrail's Technical Services Laboratory and Amtrak's Engineering Department (Mr. Nicholas Skoutelas) would share the cost of slow bend testing. It was hoped that enough information would be gathered to then establish a mismatch specification for mobile flash butt welding.

Production of Test Welds

The test welds were produced on May 16, 1996, at Brandywine, MD, on Conrail's Pope's Creek Secondary line. Weather conditions were 58°F with a light mist. The rain stopped shortly after the welding began. Three different rail sections were used to produce a total of nine welds. Fit 1951 132 RE CC Illinois rails with 1/8 inches head loss (7 inches section height) were selected at Lucknow and transported to the site. Also sent to Brandywine were new 1991 132 RE CC Bethlehem rails (7 1/8 inches section height) and new 1996 136-10 CC Bethlehem medium head hardened rails (7 5/16 inches section height). For the first three welds, there was no base mismatch as 136 was welded to 136. Welds four, five and six were produced with nominal base mismatch of 5/16 inches as the fit 132 rail was welded to new 136. Finally welds seven, eight and nine involved the welding of fit 132 rail to new 132 rail which resulted in a nominal base mismatch of 1/8 inches. No base bottom grinding was performed on the welds prior to slow bend testing. Figure 3 shows the Holland mobile welding truck used to produce the test welds. In order to provide adequate electrical contact between the mobile welding head and the rail web, grinding must be performed prior to welding. Figure 4 shows the grinding operation on the rails prior to production.

Figure 3. Holland mobile welder at Brandywine, MD.

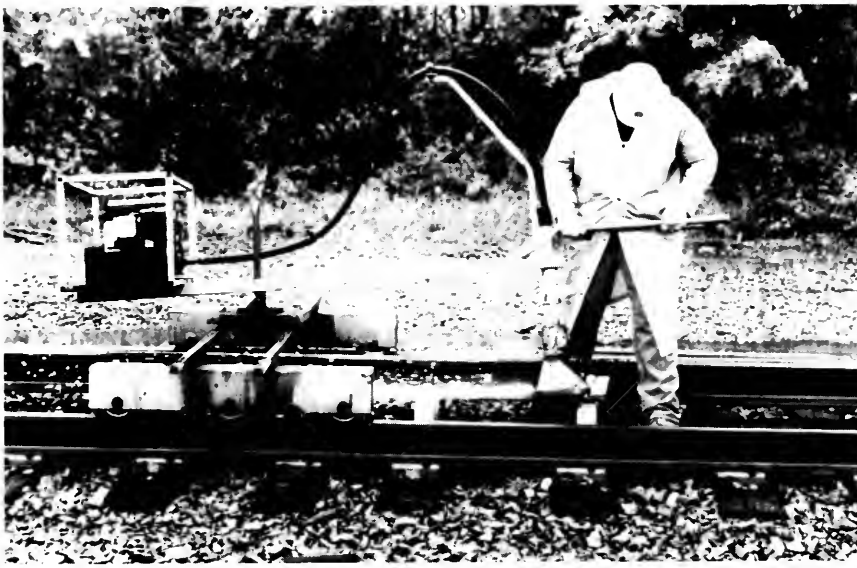

Figure 4. Pre-weld grinding of rail web.

The approximate maximum amount of rail mismatch that can be accommodated by the welding head and shear is 5/16 inches. However, during production of welds five and six, some of the mismatch (about 0.025 inches) was accounted for in the head of the rails. As the welding machine head forged the two rail ends together, some movement occurred which lessened the amount of base mismatch. Weld four had closer to the desired 5/16 inches of base mismatch. Figure 5 shows the mobile welding head, suspended by a boom off the back of the truck, in operation during weld production. Holland's Mr. Ken Merritt is at the controls while Mr. Ernie Mills looks on. Figure 6 shows test weld four and the large amount of base mismatch.

Slow Bend Testing Results

The nine test welds were transported to the Fritz Engineering Laboratory at Lehigh University in Bethlehem, PA, for slow bend testing. The machine used for testing was an 800,000 pounds Riehle screw type machine and the tests were conducted in accordance with A.R.E.A. procedure specifications. Welds one, two, five, seven and nine reached two inches of deflection without failure and the tests were arbitrarily stopped at this point after noting the load. Modulus of rupture was then calculated using the appropriate rail base section modulus value and the equation in the A.R.E.A. Manual (Ref. 5). All values are shown in Table 1.

The only problem encountered during the slow bend tests was seen during the testing of weld number seven. There was a sudden drop in load at 394.7 kips and 1.27 inches deflection, accompanied by a loud noise. This was most likely due to the shifting of a test fixture. No crack was seen in the weld or rails and the sample eventually reached excellent modulus of rupture and deflection values as shown in Table 1.

Discussion of Results

The results in table one show that all welds are classified as acceptable welds when compared to the A.R.E.A. slow bend test load and deflection requirements (Ref. 4). Standard carbon rail (300

Figure 5. Mobile Holland welder in operation.

Figure 6. Test weld number four.

Table 1. Results of slow bend testing at Lehigh University.

Weld Number	Mismatch (in.)	Load (kips)	Modulus Of Rupture (ksi)	Base Section Modulus Used (cu. in.)	Deflection (in.)	Break
1	0	513.8	163.98	28.2	2	No
2	0	518.7	165.54	28.2	2	No
3	0	472.8	150.89	28.2	1.49	Yes
Average Value		**501.8**	**160.14**	—	**1.83**	—
4	5/16	390.3	128.2	27.4	1.22	Yes
5	5/16	449.4	147.6	27.4	2	No
6	5/16	414.3	136.1	27.4	1.345	Yes
Average Value		**418**	**137.3**	—	**1.52**	—
7	1/8	458.2	150.5	27.4	2	No
8	1/8	370.9	121.8	27.4	1.15	Yes
9	1/8	464	152.4	27.4	2	No
Average Value		**431**	**141.6**	—	**1.72**	—

BHN minimum) is required to have a modulus of rupture of at least 120,000 psi and a deflection of at least one inch. High strength rail (341 BHN minimum) is required to have a modulus of rupture of at least 125,000 psi and a deflection of at least 0.75 inches. The lowest modulus of rupture value and deflection value is seen for weld eight which had values of 121.8 ksi and 1.15 inches. This weld had a mismatch of 1/8 inches at the base. The weld with the highest values was weld two (no mismatch) with values of 165.5 ksi and 2 inches.

A review of the values in Table 1 shows that the welds with no mismatch have the highest load, modulus of rupture and deflection average values. Welds with 1/8 inches mismatch have the next highest average values while welds with the most mismatch (nominal 5/16 inches) had the worst average values for load, modulus of rupture and deflection. This is reasonable since a greater degree of mismatch at the base provides a larger notch for stress to be concentrated during loading. A greater degree of stress concentration should logically lead to failure at lower levels of load and deflection. An increase in the amount of rail base mismatch does therefore degrade the slow bend properties of the welds.

Given the small sample size for each weld group (3 welds for each group) it is helpful and necessary to calculate 95% confidence intervals for the properties of each weld group. This information is contained in Table 2.

The data shows that there is significant overlap in the confidence intervals between the three weld groups for each parameter. With the small sample size of the three test welds for each amount of rail base mismatch, this is not surprising. It is expected that the 95% confidence intervals would narrow as the number of welds tested increased.

An examination of the fracture surfaces of the four weld samples which failed revealed that fast fracture originated at identifiable points in each case. Crack chevrons indicated the origins. Weld number three's (no mismatch) fracture originated at small, shiny, finer areas on the fracture surface in the base of the rail. These areas on flash weld fracture surfaces have been called "flat spots" in the literature (Ref. 6, 7, 8). Several were clearly visible in the foot of the rail. The largest one measured approximately 1/16 inches wide by 1/4 inches long and was oriented vertically. The break occurred approximately in the center of the weld. Weld number four's (5/16 inches mismatch) fracture started at a larger, slightly oval shaped defect in the far foot of the rail. This defect is approximately 1/4 inches in diameter, is roughly horizontal in orientation and is near the edge of the weld. The defect is an area of incomplete fusion in the weld and appears to be related to the large amount of mismatch between

Table 2. Results showing 95% confidence intervals.

Weld Group	Mismatch (in.)	Average Load (kips)	95% Confidence Interval (kips)	Average Modulus of Rupture (ksi)	95% Confidence Interval (ksi)	Average Deflection (in.)	95% Confidence Interval (in.)
1–3	0	501.8	± 28.5	160.1	± 9.1	1.83	± 0.33
4–6	5/16	418.0	± 33.6	137.3	± 11.0	1.52	± 0.47
7–9	1/8	431.0	± 59.0	141.6	± 19.4	1.72	± 0.55

the rails. Weld number six's crack chevrons point back towards a small oxide entrapment (1/8 inches in diameter) in the toe of the rail near the edge. The fracture surface also exhibited "flat spots" in the base and just above the base of the rail in the web. Finally, weld number eight's (1/8 inches mismatch) fracture origination point was in the extreme edge of the rail base. A semi-circular defect 1/8 inches long and 1/16 inches wide was seen which appeared to be an oxide entrapment in the weld at the edge.

For the four weld samples we tested to failure, the worst slow bend properties were seen for the three welds with obvious fracture surface defects. Better properties were seen for the weld with only "flat spots" on the fracture surfaces although these properties were nowhere near the levels seen for the welds which did not break. It is therefore seen in our test that flat spots were not as damaging to mechanical properties as the oxide entrapment or incomplete fusion defects. Flat spots were seen to be associated with reduced ductility and reduced resistance to cracking in previous experiments with small flash welded steel specimens (Ref. 7). It has been suggested that "flat spots" are caused by oxygen/atmosphere contamination or carbon segregation leading to localized areas of martensite (Ref. 7, 8). Shaw concluded that flat spots in rail welds were caused by large flash craters which were not completely forged out (Ref. 6).

It must be noted that the dynamic properties of a rail weld in track are far different than the properties measured in a slow bend test. With each wheel of a passing train the rail weld experiences one fatigue cycle, with the maximum tensile stress occurring at the bottom of the rail base. If a weld has a large degree of rail base mismatch, a larger notch will be present which provides a stress concentration. Also, due to the difficulty in shearing welds with large amounts of mismatch after weld production, discontinuities, notches and gouges can result at the base. Under the cyclic tensile loading of train movements, localized yielding of the steel at these discontinuities will initiate fatigue cracks. Further such loading by trains will result in crack propagation and potential failure. The tests we conducted merely illustrated how well the weld samples met the A.R.E.A. slow bend test specification and how well the samples performed relative to each other.

Conclusions

1. All welds tested met the specified A.R.E.A. values for modulus of rupture and deflection of electric flash butt rail welds in a slow bend test.

2. Two welds with no rail base mismatch reached two inches of deflection without failure along with two rails having 1/8 inches mismatch and one having 5/16 inches mismatch.

3. The welds with no rail base mismatch had the highest average load, modulus of rupture and deflection values. Welds with 1/8 inches mismatch had the next best average values and welds with 5/16 inches mismatch had the worst average values.

4. There is significant overlap of 95% confidence intervals for each average load, modulus of rupture and deflection value among weld groups. This is likely due to the small sample size of three welds per group.

5. Three of the four welds tested to fracture had obvious defects on their fracture surfaces. One of these welds was made with 1/8 inches mismatch and the other two were produced with 5/16 inches mismatch. Two welds, one with no mismatch and the other with 5/16 inches mismatch, exhibited "flat spots" on their fracture surface. The latter weld also had an obvious defect present on its fracture surface. The welds with obvious defects had lower load, modulus of rupture and deflection values than the weld with only "flat spots" on the fracture surface.

Recommendations

1. All welds produced by the mobile in track welder met the required A.R.E.A. values for modulus of rupture and deflection, including those welds produced with the maximum base mismatch allowed by the welding head. Given this information no prohibition or limit on the amount of rail base mismatch, other than the physical limit of the machine, is needed. However, railroads must be aware that local conditions and heavy train traffic will influence the ability of these joints to withstand cyclic loading.

2. For best weld integrity, base matching is preferred. However, the results of our limited testing have demonstrated that welds with an amount of base mismatch up to the limit of the in track welding head produces a weld which meets the established A.R.E.A. criteria for electric flash butt welds.

3. Current railroad practice of closely matching rail sections prior to welding should continue. This will help minimize the chance for fatigue initiations over the life of the weld.

4. We recommend that local track supervisors set aside "special" rail sections for repair of heavily curve worn areas, etc. This will provide best rail matching and weld performance.

5. An investigation should be conducted to determine the origin and composition of the "flat spots" noted on the weld fracture surfaces. It should also be determined to what degree they are detrimental to rail weld static and dynamic properties.

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Memoir
Thomas J. Lamphier
1924–1996

Born May 2, 1924 in Rochester, NY, Tom Lamphier graduated from Massachusetts Institute of Technology with a Bachelor of Science Degree in Civil Engineering in 1949. He began his railroad career with the Great Northern upon graduation. He began in the maintenance-of-way department where he served in various postions including that of Roadmaster. He moved into transportation and had a 3-year stint as Assistant Trainmaster, after which he became co-chairmen of the Computer Research Committee in 1956 at GN headquarters in St. Paul. This launched Tom Lamphier into a career of management information systems and led to executive positions with GN. In 1963, he was Division Superintendent at Klamath Falls, OR to get re-acquainted with "real" railroading. A year later he was appointed Assistant to Vice President Operations, and in 1970 Tom Lamphier began a 7-year stretch of heading various executive functions, becoming Executive Vice President of Burlington Northern in 1976. One year later, he became President of BN Transportation (basically President of the railroad).

Over the years, Tom Lamphier was a very active member of AREA Committee 16, and made many contributions to Committee reports and to various revisions of Chapter 16 of the Manual.

Upon retirement, he was active in getting MIT fully involved in rail transportation research and study. He died on September 9, 1996.

He will be greatly missed, not only by his friends, but for his contributions to AREA Committee 16.

Note: This article was written by Robert W. McKnight, a former secretary of Committee 16 during the 1960's and early 1970's.

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