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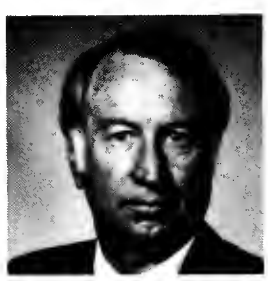

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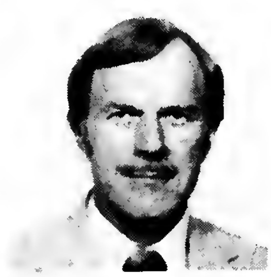

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January 1984 #694

Eastbound freight train about 2 miles west of Xian, China, station, May 1983. Local freight is in background.

May 1984 #696

Eastbound commuter train heads for Montreal on Canadian Pacific mainline west of Dorval, Quebec. Canadian National mainline is at left.

October 1984 #697

Eastbound unit coal train on Burlington Northern's Lincoln-Alliance, Nebraska Line west of Ravenna, Nebraska July 3, 1984.

December 1984 #698

Ferrocarriles Nacionales de Mexico employees with special A.R.E.A. train from Mexico City to Queretaro wait at Tula Station while delegation visits bridge projects nearby, October 26, 1984.

Numerical Index to Committee Reports

Additional committee reports are contained in AREA News section of Railway Track and Structures magazine December 1983 through March 1984 issues.

American Railway Engineering Association

Bulletin 694

JANUARY 1984

Proceedings Volume 85 (1984)

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Front Cover Photo: Eastbound freight train about 2 miles west of Xian, China, station, May 1983. Local freight is in background.

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Railroading In Xian, China

Photos of trains in action taken on the technical visit of A.R.E.A. members to China, May 1983

While the A.R.E.A. *Bulletins* are normally limited to subjects from Canada, Mexico, and the U.S.A., it was felt that readers would enjoy action photos from the 1983 visit to China. (Technical aspects will be covered at the March A.R.E.A. conference and published in Bulletin 696). Railroading in and around the city of Xian was selected for this cover feature. While Chinese motive power is approximately 20% electric and diesel, Xian was 100% steam at the time of our visit. The photo above shows a QJ-class 2-10-2 locomotive (a type still being built in China) coming into the Xian station from the east with a fifteen car passenger train, while the photo below shows a freight heading east at the same location.

The photo above was taken from the train on which the delegation was traveling from Sanmenxia to Xian behind an RM-class 4-6-2 locomotive. The shot shows the terracing to create farmable land on the hillsides. The shot below shows a similar locomotive and train heading east about one mile west of Xian station. The delegation of A.R.E.A. members was royally treated on its visit to this scenic and fascinating country, with superb meals at hotels and in dining cars, and the friendship of many Chinese railway engineering professionals as we shared our technical knowledge. Chinese railways are standard gauge and use the same basic coupling and air brake systems as in North America.

The Chinese railways haul the vast majority of both intercity ton-miles and passenger-miles in that country, and the network of railway lines is not dense, creating high traffic situations. The double track line through Xian had 162 through trains per day at the time of our visit, 82 passenger and 80 freights. Local switching moves were in addition to this. The above photo shows an eastbound passenger train about five miles west of Xian and that below a westbound freight near the same location.

Comments on the Manual and Portfolio revisions shown on page 5 through 66 are welcomed from all readers and should be received at Headquarters by March 1, 1984.

MANUAL RECOMMENDATIONS

COMMITTEE 1—ROADWAY AND BALLAST

The Committee recommends a major change for Manual Sections 3.1 through 3.3, pages 1-3-1 through 1-3-8. This change concerns methods for determining the flood flows that the waterway or waterway opening must be capable of passing successfully.

This revision enlarges coverage of the methods of flood flow prediction. It provides many more details of the use of the Rational Method including a step-by-step delineation of the assembly of the numbers (from the accompanying Tables) and their manipulation.

Statistical methods are presented for those locations where sufficient flood flow data are available. This includes a step-by-step delineation of the use of the Log Pearson Type III distribution that is mandated by Federal regulations.

Also covered is the Soil Conservation Service Curve Number method, a procedure that is widely becoming accepted for use. The use of this method is outlined in detail, the necessary data are presented in accompanying Tables, and the assembly of the numbers is detailed in a step-by-step formulation to obtain both total flow and peak flow from a specific storm.

The theme of this proposed revision is to place in the Manual a self-contained development of procedures and data to allow flood flow prediction in a modern manner using accepted procedures.

Copies of the full text of these proposed Manual changes are available from headquarters at the cost of $2.00 for this document.

The Committee recommends that Chapter 1, Part 4, Culverts, be revised as indicated in the following synopsis:

Part 4 — Culverts, includes 72 pages of material specifications and design requirements for corrugated metal pipe culverts. Part 4 also includes a new decimal numbering system for clarity and ease in reading.

Section 4.1 includes a discussion on waterway, span, character of hydraulic traffic, topographic conditions, foundation conditions, live and dead loading and economics of various pipe types. These paragraphs remain essentially the same as in previous editions of the Manual.

Section 4.5 begins with material specifications for prefabricated corrugated steel pipe and pipe-arches for culverts, storm drains, and underdrains. This part of the specification describes requirements for annular corrugations, helical corrugations, circular pipe, elongated pipe and pipe-arch cross sections.

Paragraph 4.5.2 "Material" has been changed to make it current with pipe material specified in the AASHTO specifications. These include both zinc coatings and aluminum coatings and are cross referenced to the AASHTO M218 specification for zinc coated steel sheets and AASHTO specification M274 for aluminum coated steel sheets.

Paragraph 4.5.3 "Fabrication" has been rewritten to include additional corrugation sizes than have previously been shown in the manual. The corrugations described in Table 1 include sizes 2-½ × ¼, 2-⅔ × ½, 3 × 1 and 5 × 1 as the available profiles for factory-fabricated corrugated steel pipe. Corrugation sizes and details for perforations for underdrains remain as previously shown in the manual.

In Paragraph 4.5.3.3, a new Table 3 has been added for pipe requirements for diameters beginning as low as 8″ through the full size range to 120″. Diameter control has been added to Table

3 to allow a maximum of ½″ or 1% of the nominal diameter as an allowable tolerance. Additionally, the maximum difference in diameters of abutting pipe ends are not to exceed ½″.

Following the requirements for riveted seams, requirements are included for helically corrugated welded-seams and helically corrugated lockseams. There are no major changes in these two types of fabrications. The requirements for resistance spot welded seams, as previously specified, are not included in the text at this time. Coupling band specified in Paragraph 4.5.4 includes provisions for field joints for each of the different classes and shapes of corrugations. Band couplers can be provided for helical pipe with reformed annular ends or for ends of pipe that remain helically corrugated.

The pipe-arch shape designated as Shape 3 is presented in Tables 5 and 6 and includes dimensions and tolerances for all sizes beginning at 17″ × 13″ through 142″ × 91″. The tables designate the corrugation profile, the span and the rise and the allowable tolerances. These numbers now match those recommended by the industry and those published by other national agencies.

Section 4.6 "Specifications for Coated Corrugated Metal Pipe and Arches" includes requirements for both bituminous-coated and polymeric-coated corrugated steel pipe. The requirements have been cross referenced to AASHTO specification M190 and AASHTO specification M246 rather than reprinting the complete specification.

Section 4.7 "Standard Specification for Corrugated Aluminum Alloy Pipe" is a new section in the specification and follows the same format and outline as used for the corrugated steel pipe product. The sections covering corrugated aluminum alloy pipe have been modified to reflect differences in fabrication procedures for the aluminum alloy product. There are no major differences between these requirements and those published by other national agencies. The aluminum specification includes requirement for both riveted and helical-type corrugations; and for Class I (annular), includes a 6 × 1 corrugation—a size not available in the corrugated steel pipe product.

Section 4.8 "Installation of Pipe Culverts" and Section 4.9 "Jacking Culvert Pipe Through Fills" are presented as previously specified in the Manual and include such major provisions as preparation of foundation, backfill, kind of pipe suitable for jacking, and a jacking procedure. These sections all should be familiar to users of the current Manual in these specialized areas.

Section 4.12 "Design Considerations of Culvert Pipes" has been added to Part 4 of Chapter 1. The format is the same as published by other major specifying agencies but has been modified somewhat to reflect the loadings associated with the railroad industry. Design requirements for both steel and aluminum are blended together into a common design section for all culvert pipe. Requirements and formulas are given for seam strength, handling and installation strength, wall area, buckling, and dead and live load requirements. For each metal, values are tabulated for specified thickness, mechanical properties, riveted seam strengths, and sectional properties. Based on allowable safety factors in Paragraph 4.12.8, height of cover tables are tabulated in separate tables for steel corrugated pipe and aluminum alloy corrugated pipe. Table 10 includes both steel and aluminum arch pipes and the minimum allowable thickness by corrugation size for each type of metal.

Section 4.13 "Specifications for Corrugated Structural Steel Plate Pipe, Pipe-Arches, and Arches" now only includes a material specification for the steel plate, the galvanizing and threaded fasteners. Previous specifications included design requirements in this section; however, as previously mentioned, these have been moved back to a common section to include both factory-fabricated pipe and field-assembled structural plate pipe.

Section 4.14 "Specifications for Corrugated Structural Aluminum Alloy Plate, Pipe, Pipe-Arches, and Arches" has been included for the first time in the text of this specification. It follows the same format as presented for structural steel plate pipe and is similar in scope as that published by other national agencies.

Section 4.17 "Specification for Steel Tunnel Liner Plates". This section on liner plates is included as it has been previously in Part 4 — Culverts. Proper design requirements are given for the two basic types of liner plate that are available from the industry. These two basic styles include a four-flanged plate and a two-flanged plate. Table 16 gives the effective sectional properties of each of the two basic styles of plates.

It is recognized that the culvert industry is constantly changing and improving their product manufactured for the railroad industry. For that reason, Committee 1 is continually monitoring this specification for the purpose of adding new products and ideas as they are developed. Committee 1 would welcome comments and suggestions from anyone using this part of the Manual for design of drainage requirements.

Copies of the full text of this proposed Manual change are available from headquarters at the cost of $3.00 for this document.

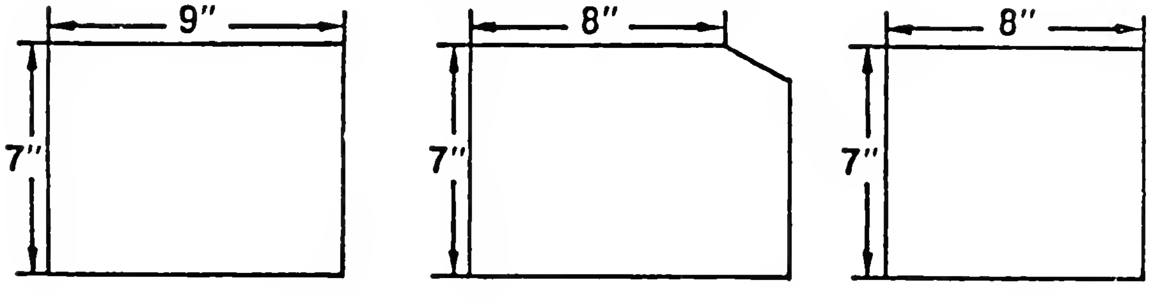
COMMITTEE 3—TIES AND WOOD PRESERVATION

The Committee recommends the following changes to Chapter 3 of the Manual: Revision of Part 1, Timber Cross Ties, and addition of new Part 11, Recommended Practice for the Manufacture of Two-Piece Steel Doweled Laminated Crossties (TPSDLC).

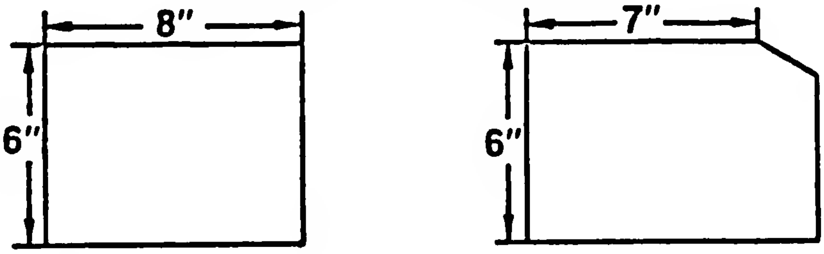
1.1.3.1 Dimensions

Ties shall be 8 ft., 8 ft. 6 in., or 9 ft. long on order of the customer. Both sections between 20 inches and 40 inches from the middle of the tie (rail bearing areas) shall measure as follows: 7" Grade ties shall be 7" x 9" in cross section with a maximum of 1" of wane in the rail bearing areas. A maximum of 20 percent 7" x 8" in cross section and no wane in the rail bearing areas will be allowed. 6" Grade ties shall be 6" x 8" in cross section with a maximum of 1" of wane permitted in the rail bearing areas.

7" GRADE CROSSTIES



6″ GRADE CROSSTIES



PROPOSED SIZE CATEGORIES FOR 7″ & 6″ CROSSTIES —
1″ OF WANE ALLOWED — 20% SQUARE 7 x 8 ALLOWED

1.1.5.3 Decay

Decay is the disintegration of the wood substance due to the action of wood destroying fungi. The following decay will be allowed: in cedar and in cypress, "pipe or stump rot" and "peck", respectively up to the limitations as to holes. "Blue stain" is not decay and is permissible in any wood.

1.1.5.5 Knots

Within the rail bearing areas, a large knot is one having an average diameter more than ⅓ the width of the surface on which it appears; but such a knot will be allowed if it is located outside the rail bearing areas. Numerous knots are any number equalling a large knot in damaging effect.

1.1.5.6 Shake

A shake is a separation along the grain, most of which occurs between the rings of annual growth.

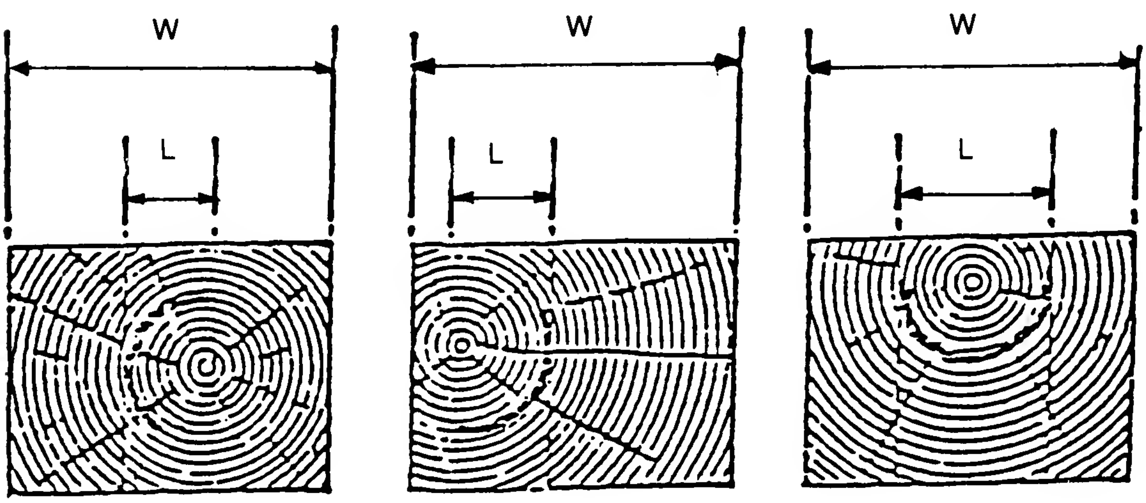


FIG. A FIG. B FIG. C

The procedure illustrated in the above diagrams shall be used in determining the length of a shake. One which is not more than ⅓ the width of the tie will be allowed, provided it does not extend nearer than 1″ to any surface.

1.1.5.7 Split

A split is a separation of the wood extending from one surface to an opposite or adjacent surface. In unseasoned crossties, a split no more than ⅛" wide and/or 4" long is acceptable. In a seasoned crosstie, a split no more than ¼" wide and/or longer than the width of the face across which it occurs is acceptable. In seasoned crossties, a split exceeding the limit is acceptable, provided split limitations and anti-splitting devices are approved by the buyer and properly applied.

1.1.5.10 Bark Seams

A bark seam or pocket is a patch of bark partially or wholly enclosed in the wood. Bark seams will be allowed provided they are not more than 2" below the surface and/or 10" long. **NOTE:** It is recommended for West Coast species that W.C.L.B. Grading Rules apply.

1.1.6.3 Grouping

To be deleted and appropriately covered in a forthcoming revision of Part 5.

1.1.6.4 Class T-Ties Which Should be Treated

To be deleted and appropriately covered in a forthcoming revision of Part 5.

1.4.3

For heavy traffic lines, ties meeting the standard specification for 7" Grade ties should be used.

1.5.5 INCISING

To be deleted from Part 1, and appropriately covered in a forth coming revision of Part 5.

1.5.7 BRANDING

(a) Branding of ends or the top or bottom surfaces of the ties as they pass through the machine shall be done with letters, figures or symbols to indicate one or more of the following:

1.9.1 ANTI-SPLITTING DEVICES

Anti-splitting devices may be of (a) the type made from a strip of steel and applied by driving into the end (cross section) of the tie; or (b) of the steel dowel type, applied parallel to the wide face of the tie, transverse to its length; or (c) the steel multi-nail plate type applied by driving into the end (cross section) of the tie.

1.9.2.3 Nail Plates

If a steel nail is used, it shall be made of at least 18 gauge galvanized steel conforming to ASTM A446 Grade 4, and the galvanizing conforming to ASTM A525.

1.10.3 NAIL PLATES

One multi-nail plate should be positioned onto the end (cross section) of the tie, with the plate being placed to cover the greatest area of splitting. This should enable the plate to hold both vertical and horizontal splits together.

Nail plates should be applied by a mechanical device capable of squeezing the splits together, bringing the tie back to its original dimensions, prior to application.

Part 11

Recommended Practice for the Manufacture of Two-Piece Steel Doweled Laminated Crossties (TPSDLC)

11.1 MATERIAL

11.1.1 KINDS OF WOOD

11.1.1.1 Before manufacturing TPSDLC's, producers shall ascertain which of the following kinds of wood suitable for crossties will be accepted:

Ashes	Cypresses	Hemlocks	Oaks	Sycamores
Beech	Douglas Fir	Hickories	Pines	Walnuts
Birches	Elms	Larches	Poplars	
Catalpas	Firs (true)	Locusts	Redwoods	
Cedars	Gums	Maples	Sassafras	
Cherries	Hackberries	Mulberries	Spruces	

*All species listed are permitted unless the buyer specifies otherwise. Density requirements on conifers, if any, to be specified by the buyer. (In eastern production areas hardwoods are recommended and should be grouped as oak and mixed hardwoods.) Each component half should be from the same species grouping, i.e., oak-oak and mixed hardwood-mixed hardwood.

11.1.1.2 Except as hereinafter provided, all pieces used to make up the TPSDLC's shall be free from any defects that may impair their strength or durability as TPSDLC components, such as decay, large splits, large shakes, large or numerous holes or knots, grain with slant greater than one in fifteen.

11.2 DESIGN

11.2.1 DIMENSIONS

11.2.1.1 Before manufacturing TPSDLC's producers shall ascertain which of the following lengths, shapes, or sizes will be accepted.

11.2.1.2 Standard gauge TPSDLC's shall be 8′0″, 8′6″, or 9′0″. The length to be specified by the buyer.

11.2.1.3 Except as hereinafter provided, TPSDLC's shall measure as follows throughout the rail bearing areas. The rail bearing areas as used here and hereafter are defined as those sections of the TPSDLC between 20″ and 40″ from its middle:

Size 5 — 7″ x 9″, minimum 9″ faces

11.2.2 GENERAL REQUIREMENTS

11.2.2.1 Except as hereinafter provided, all TPSDLC's shall be straight, well-manufactured, cut square at the ends, and have the bark entirely removed.

11.2.2.2 After doweling, all TPSDLC's shall be manufactured such that one 9″ surface shall be flat, without offset between the two components. An offset of not more than ⅛″ between the two components shall be permitted on the opposite surface. All air-seasoned TPSDLC's should be surfaced after seasoning.

11.2.3 DOWELING

11.2.3.1 Dowels shall be steel, either three of four fluted, and shall be ½" in diameter with ⅜" root diameter. Dowel lengths used shall be 8-¾" for 7" x 9" TPSDLC's.

11.2.3.2 Two dowels shall be required at a point 5 inches from either end and at the midpoint of every TPSDLC, regardless of its length, for a total of six dowels per tie. Dowel holes shall be ⅜" in diameter.

11.2.3.3 In a nominal 7" thick tie, the dowels will be inserted 4" apart, which will place them 1-½" ± ⅛" from the top or bottom of the tie.

11.3 INSPECTION

11.3.1 LOCATION

11.3.1.1 Each piece to be used in making up a TPSDLC shall be inspected before being doweled into place. These pieces will be inspected at suitable and convenient places, at point of shipment or at destination, as may be agreed between the supplier and the buyer.

11.3.1.2 Each completed TPSDLC will likewise be inspected at a suitable and convenient place, either at point of shipment or at destination, as may be agreed between the supplier and the buyer.

11.3.2 TOLERANCES

11.3.2.1 *Decay*. "Blue stain" is not decay and is permissable in any wood.

11.3.2.2 *Holes*. Within the rail bearing areas a large hole is one more than ½" in diameter and 3" deep, excepting one caused by "pipe or stump rot" in cedar. Outside the rail bearing areas a large hole is one having a diameter more than ¼ the width of the surface on which it appears and a depth of more than 1-½". Numerous holes are any number equaling a large hole in damaging effect. Such holes may be caused in manufacture or otherwise.

11.3.2.3 *Knots*. Within the rail bearing areas a large knot is one having an average diameter more than ⅓ the width of the surface of the component on which it appears; but such a knot will be allowed if it is located outside the rail bearing areas. Numerous knots are any number equaling a large knot in damaging effect.

11.3.2.4 *Shakes*. Shakes are acceptable provided largest dimension measuring length is not more than ⅓ of width and provided they do not extend nearer than 1" to any surface. The procedure illustrated in diagrams shall be used in determining the length of a shake.

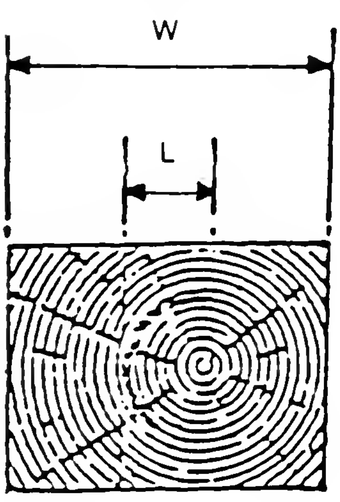


FIG. A

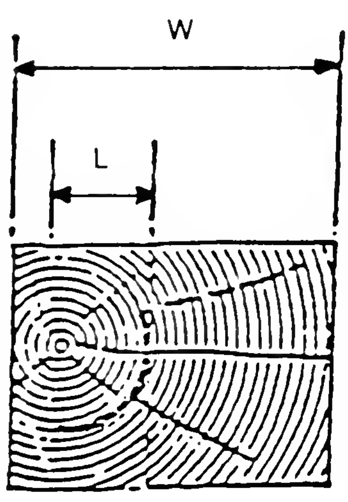


FIG. B

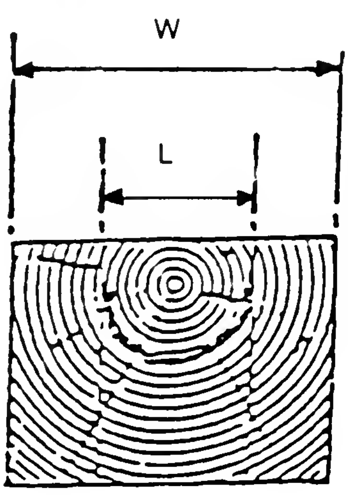


FIG. C

11.3.2.5 Splits

11.3.2.5.1 A split is a separation of the wood extending from one surface to an opposite or adjacent surface. In a TPSDLC component, a split no more than ⅛″ wide and/or 4″ long is acceptable.

11.3.2.5.2 Anti-splitting devices will not be allowed on component pieces which have splits exceeding these limits. Such pieces are deemed unacceptable and shall not be used in manufacturing a TPSDLC.

11.3.3 MANUFACTURE

11.3.3.1 All TPSDLC's will be manufactured from components cut from live trees. A component will be considered straight: (1) when a straight line along the top from the middle of one end to the middle of the other end is everywhere at least 1″ from the edge of the component, and (2) when a straight line along a side from the middle of one end to the middle of the other end is everywhere more than 2″ from the top and bottom of the component.

11.3.3.2 A TPSDLC is not well manufactured when its surfaces are cut into with score marks more than ½″ deep.

11.3.3.3 The top and bottom of the TPSDLC will be considered parallel if any difference in the thickness at the sides or ends does not exceed ½″.

11.3.3.4 Dimensions

11.3.3.4.1 Specified dimensions for TPSDLC's apply to the unseasoned condition. Specified thickness and widths are considered to be met after conditioning if the TPSDLC's are not more than ¼″ thinner or narrower than the specified sizes. TPSDLC's over 1″ thicker or wider than the specified sizes may be rejected. TPSDLC's over 2″ longer or 1″ shorter than the specified lengths may be rejected.

11.3.3.4.2 Minimum unseasoned component size shall be a full 4-½″ x 7″.

11.3.3.4.3 All thickness, width, and face dimensions, apply to the rail bearing areas of the TPSDLC. All determinations of the width will be made on top of the TPSDLC, which is the narrower of the horizontal surfaces, or the one with the narrower or no heartwood if both horizontal surfaces are of the same width.

11.3.3.4.4 Wane appearing anywhere along the joint between components is cause for reject. A maximum of 1″ wane will be permitted on outside corners not within the rail bearing areas.

11.3.3.4.5 There must be a tight fit between components. Warp on only one component will be permitted provided it does not exceed ⅛″ from a straight line after doweling.

11.4 DELIVERY

Place and manner of delivery to be as agreed between supplier and buyer.

11.5 SHIPMENT

Means and manner of shipment to be as agreed between supplier and buyer.

11.6 TIE PLATES

AREA B punch plates are not recommended. Smooth-bottom plates only should be used with TPSDLC's.

COMMITTEE 4—RAIL

The Committee recommends the following revisions to Chapter 4, Part 2, Specifications, of the Manual:

SPECIFICATIONS FOR STEEL RAILS

1. Scope

1.1 These specifications cover steel tee rails for use in railway track.

1.2 Supplementary requirements S1 through S3 shall apply only when specified by the purchaser.

2. Manufacture

2.1 The steel shall be made by any of the following processes: open hearth, basic oxygen, or electric furnace.

2.2 The steel shall be cast by a continuous process, in hot topped ingots, or by other methods agreed by purchaser and manufacturer.

2.3 Sufficient discard shall be taken from ingots and blooms rolled from ingots to insure freedom from injurious segregation and pipe.

3. Chemical Composition

3.1 The chemical composition of the standard rail steel determined as prescribed in 3.3 shall be within the following limits:

	Chemical Analysis Weight Percent Nominal Weight lb/yd		Product Analysis Weight Percent Allowance Beyond Limits of Specified Chemical Analysis	
Element	90 to 114	115 & Over	Under Min.	Over Max.
Carbon	0.67-0.80	0.72-0.82	0.04	0.04
Manganese	0.70-1.00	0.80-1.10*	0.06	0.06
Phosphorus, Max.	0.035	0.035	—	0.008
Sulfur, Max.	0.037	0.037	—	0.008
Silicon	0.10-0.50	0.10-0.50	0.02	0.02**

*The upper manganese limit may be extended to 1.25% by the manufacturers to meet the hardness specifications. When the manganese exceeds 1.10% the residual alloy contents will be held to 0.25% max. Ni, 0.25% max. Cr, 0.10% Mo.

**Continuously cast tolerances shall be 0.05% over maximum limit for Silicon.

3.1.1 Finished material representing the heat may be product tested. The product analysis shall be within the limits for product analyses specified in the Table of 3.1.

3.2 The chemical composition of alloy high strength rail will be subject to agreement of the purchaser and manufacturer.

3.3 Separate analysis shall be made from test samples representing one of the first three and one of the last three ingots or continuously cast blooms preferably taken during pouring of the heat. Determination may be made chemically or spectrographically. Any portion of the heat meeting the chemical analysis requirements of 3.1 may be applied. Additionally, any material meeting the product analysis limits shown in 3.1 may be applied after testing such material.

3.4 Upon request by the purchaser, samples shall be furnished to verify the analysis as determined in 3.3.

3.5 The first analysis shall be recorded as the official heat analysis, but the purchaser shall have access to all chemical analysis determinations.

4. Hardness Properties

4.1 Rails shall be produced as specified by the purchaser as within the following limits:

	Standard Rail		High Strength Rail
	90-114 lb./yd.	115 and over lb./yd.	
Brinell Hardness	248 min.	269 min.	321-388

4.2 A Brinell hardness test shall be performed on a rail or a piece of rail at least 6 inches long cut from a rail of each heat of steel and a report furnished to the purchaser.

4.2.1 The test shall be made on the side or top of the rail head, after decarburized material has been removed, to permit an accurate determination of hardness.

4.2.2 The test shall otherwise be conducted in accordance with the American Society of Testing and Materials (ASTM) Standard Method of Test for Brinell Hardness of Metallic Materials E10 latest version.

4.3 If any hardness test fails to meet the specifications, two additional checks shall be made, one on each of the point first measured. If both checks meet the specified minimum hardness as ordered, the heat shall have met the hardness requirement. If either of the additional checks fails, two further rails in the heat shall be checked with each of these two rails meeting the minimum ordered for the heat to be accepted. If any one of these two checks fails, individual rails may be tested for acceptance.

4.4 If for heat treated rails a test fails to meet the requirements of 4.1, the rails may be retreated, at the option of the manufacturer, and such rails may be retested in accordance with 4.2 and 4.3.

5. Section

5.1 The section of the rails shall conform to the design specified by the purchaser subject to the following tolerances on dimensions:

	Inches (Thousandths) Plus	Minus
5.1.1 height of rail (measured within 1 ft. from end)	0.030	0.015
5.1.2 width of rail head (measured within 1 ft. from end)	0.030	0.030
5.1.3 thickness of web	0.040	0.020
5.1.4 width of either flange	0.040	0.040
5.1.5 width of base	0.050	0.050

5.1.6 No variation will be allowed in dimensions affecting the fit of the joint bars, except that the fishing templet may stand out not to exceed 0.060″ laterally.

5.2 Verification of tolerances shall be made using appropriate gages, as agreed upon by purchaser and manufacturer.

6. Branding and Stamping

6.1 Branding shall be rolled in raised characters on the side of the web of each rail at a minimum of every 16 ft. in accordance with the following requirements:

6.1.1 The data and order of arrangement of the branding shall be as shown in the following typical brand, the design of letters and numerals to be optional with the manufacturer.

132 (Weight)	RE (Section)	CC (Method of Hydrogen Elimination if indicated in Brand)	Manufacturer (Mill Brand)	1982 (Year Rolled)	III (Month Rolled)

6.2 The web of each rail shall be hot stamped at a minimum of every 16 ft. on the side opposite the brand in accordance with the following requirements:

6.2.1 The data shall be shown in the following typical stamping. The height of the letters and numerals shall be ⅝″.

297165 (Heat Number)	ABCDEFGH (Rail Letter)	12 (Ingot Number) or (Strand & Bloom Number)	BC (Method of Hydrogen Elimination, if indicated in stamping)

6.2.2 The top rail from each ingot shall normally be hot stamped "A" and succeeding ones "B", "C", "D", "E", etc., consecutively.

6.2.2.1 The top rail from each hot topped ingot may be hot stamped "B" and succeeding ones "C", "D", "E", etc. consecutively, when agreed between purchaser and manufacturer.

6.2.3 Ingots shall be numbered in the order cast.

6.2.4 Rails from continuous cast blooms shall be identified by a designation for heat number, strand number, and bloom number.

(Note strand and bloom numbers may be joined or may be coded at the manufacturer's option.)

The rail shall be identified by an alphabetical designation beginning with "P", and succeeding "R", "S", "T", etc., consecutively, or any other identification of the position of the rail within the cast, as agreed between the purchaser and manufacturer.

6.2.5 Stamping shall be legible and not injurious to the rail. The characters shall be of a uniform depth and approximately centered on the web.

6.2.6 High strength rail shall be identified in accordance with Section 15.1.

7. Hydrogen Elimination

7.1 The rail shall be free of shatter cracks.

7.2 The above shall be accomplished by a least one of the following processes:

Control Cooling of Rails (CC) (See Appendix 1)
Control Cooling of Blooms (BC)
Vacuum Treated (VT)
Such other processes as will meet the conditions of 7.1 (OP)

7.3 The mill brand or stamp shall identify the process used by the initials in parenthesis shown in Section 7.2.

8. Resistance to Impact

8.1 Rail produced by a continuous casting process is not subject to this requirement.

8.2 Resistance to impact shall be determined on a machine which conforms to the requirements ot the AREA "Specifications For Drop Test Machine."

8.3 Test Specimens

8.3.1 Drop tests shall be made on test specimens of rail not less than 4 ft. and not more than 6 ft. in length.

8.3.2 The test specimens shall be cut from the top of the top rail from one of the first three, one of the middle three, and one of the last three ingots of each heat.

8.3.3 Temperature of the test specimens shall not exceed 100°F.

8.4 Test Procedure

8.4.1 The distance between supports shall be 3 ft. for sections under 106 lbs. For sections 106 to 140 lbs., it shall be 4 ft. For sections over 140 lbs., it shall be 4 ft., 8 in.

8.4.2 The test specimens shall be placed head upwards on the supports and subjected to one blow from the tup falling free from the following heights for rails of the nominal weights indicated.

Weight per Yard Pound	Feet
90-100	19
101-120	20
121 & Over	22

8.5 Test Requirements

8.5.1 If all three specimens withstand the above drop test without breaking between the supports, all of the rails of the heat will be accepted subject to final inspection for surface, section, finish, and testing for internal imperfections.

8.5.2 If any specimen breaks in a location other than between the supports, or due to a defective test specimen or drop testing machine malfunction, the test shall be disregarded and a retest shall be taken from the top of the rail involved.

8.5.3 If one of the three specimens fails, subject to the requirements of 8.4.2, all of the top rails of the heat shall be rejected.

8.5.4 Specimens shall then be cut from the bottom end of the same top rails or the top end of the "B" rails of the same ingots and tested subject to 8.4.2. If any of these specimens fails, the "B" rails of the heat shall be rejected.

8.5.5 Three additional specimens shall then be taken from the bottom end of the "B" rails or the top end of the "C" rails of the same ingots and tested subject to 8.4.2. If none of these specimens fail, the balance of the heat shall be accepted subject to final inspection for surface, section and finish. If any of the specimens fail, the entire heat shall be rejected.

8.5.6 Test provisions for rails longer than 39 ft. shall be as agreed between purchaser and manufacturer.

9. Interior Condition

9.1 For rails from ingots, a test piece representing the top end of the top rail of each ingot of each heat rolled, which has passed the drop test requirement of Section 8, shall be nicked and broken. If the fracture on any test specimen exhibits seams, laminations, cavities, evidence of injurious segregation, or interposed foreign matter, the heat number and ingot number shall be recorded and the top end and bolt holes of the finished rail, so recorded, shall be closely examined for those defects. If the finished rail is clear of the above defects when presented for inspection, it shall be accepted, subject to the requirements of 10. If the finished rail shows defects, it shall be broken or cut back through successive rails to sound metal and accepted as a short rail, subject to the requirements of 10 and 11.

9.2 Short rails produced under this procedure shall be excluded from consideration in the limitation of 11.2.

9.3 Paragraph 9.1 may be waived for ingot cast steel if the purchaser and manufacturer agree to ultrasonic testing in accordance with S.2.1.

9.4 Rails rolled from continuously cast blooms shall be evaluated for interior soundness by macro-etching in a hot acid solution.

9.4.1 One full section transverse rail specimen representing each strand of each heat cast shall be prepared and etched in a hot acid solution in order to reveal the macrostructure.

9.4.2 If any specimen fails to meet the macro-etch rail standard for interior quality, as agreed upon by the purchaser and manufacturer, two further samples of rail representative of the same strand shall be obtained.

9.4.3 These retests shall be taken one from each side of the original sample at positions selected by the manufacturer and the material from between the two retest positions shall be rejected.

9.4.4 If any retest fails, testing shall continue until acceptable internal quality is exhibited.

9.4.5 All rails represented by failed tests shall be rejected.

9.5 Rails produced by a continuous casting process are not subject to the requirements of 9.1 and 9.2 for ingot cast steel.

10. Surface Classification

10.1 Rails which do not contain surface imperfections in such number or of such character as will, in the judgement of the purchaser, render them unfit for recognized uses, shall be accepted.

11. Length

11.1 The standard length of rails shall be 39 ft. and/or 78 ft. when corrected to a temperature of 60°F as specified by the purchaser.

11.2 Up to 15 percent of 78 ft. or 9 percent of 39 ft. rail of the total tonnage accepted from each individual rolling will be accepted in shorter lengths as follows: 77′ - 76′ - 75′ - 70′ - 65′ - 60′ - 38′ - 37′ - 36′ - 33′ - 30′ - 27′ - 25′.

11.3 A variation of plus or minus 7/16 in. on 39 ft. rails or plus or minus 7/8 in. on 78 ft. rails from the specified length will be permitted.

11.4 Standard short length variations other than those set forth in 11.2 and 11.3 may be established by agreement between the purchaser and manufacturer.

11.5 Lengths of rails shall be designated with proper color paint as set forth in Section 15.

12. Drilling

12.1 The purchaser's order shall specify the amount of right-hand drilled and left-hand drilled rails, drilled-both-end rails and undrilled (blank) rails desired. The right-hand or left-hand end of the rail is determined by facing the side of the rail on which the brand (raised characters) appears.

12.1.1 When right-hand and left-hand drilling is specified, at least the minimum quantity of each indicated by the purchaser will be supplied.

12.1.2 Disposition of short-rails which accrue from left-hand drilled, right-hand drilled, and undrilled (blank) rail production, and which are acceptable in accordance with 11.2 shall be established by agreement between the purchaser and the manufacturer.

12.2 Circular holes for joint bolts shall be drilled to conform to the drawings and dimensions furnished by the purchaser.

12.2.1 A variation of nothing under 1/16 in. over in the size of the bolt holes will be permitted.

12.2.2 A variation of 1/32 in. in the location of the holes will be permitted.

12.3 Fins and burrs at the edge of bolt holes shall be eliminated. The drilling process shall be controlled so as not to mechanically or metallurgically damage the rail.

13. Workmanship

13.1 Rails shall be straightened cold in a press or roller machine to remove twists, waves and kinks until they meet the surface and line requirements specified, as determined by visual inspection.

13.2 When placed head up on a horizontal support, rails that have ends higher than the middle will be accepted, if they have a uniform upsweep, the maximum ordinate of which does not exceed 3/4″ in 39 ft. as illustrated in Fig. 1.

TOLERANCES FOR INSPECTION OF RAIL

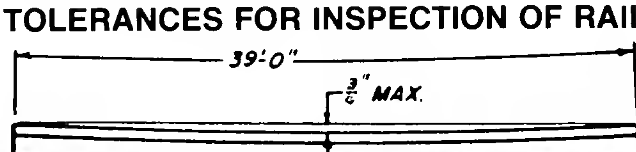


FIG. 1. Side Elevation of Rail Uniform Upsweep Tolerance per Section 13.2

13.3 The uniform surface upsweep at the rail ends shall not exceed a maximum ordinate of 0.025″ in 3 ft. and the 0.025″ maximum ordinate shall not occur at a point closer than 18″ from the rail end as illustrated in Fig. 2.

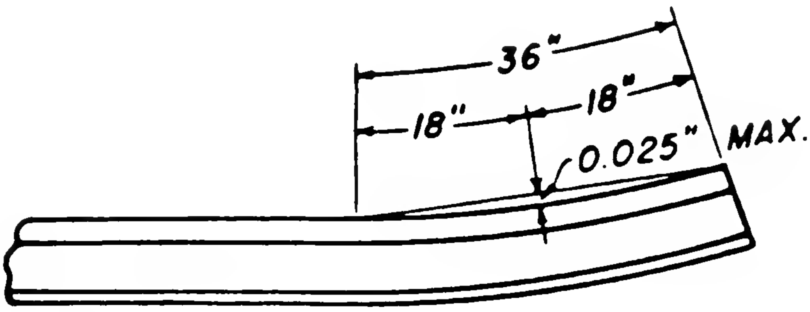


FIG. 2. Side Elevation of Rail Uniform Upsweep Tolerance at Rail Ends per Section 13.3

13.4 Surface downsweep and droop shall not be acceptable.

13.5 Deviations of the lateral (horizontal) line in either direction at the rail ends shall not exceed a maximum mid-ordinate of 0.030 in. in 3 ft. using a straight edge and of 0.023 at the end quarter-point as illustrated in Fig. 3.

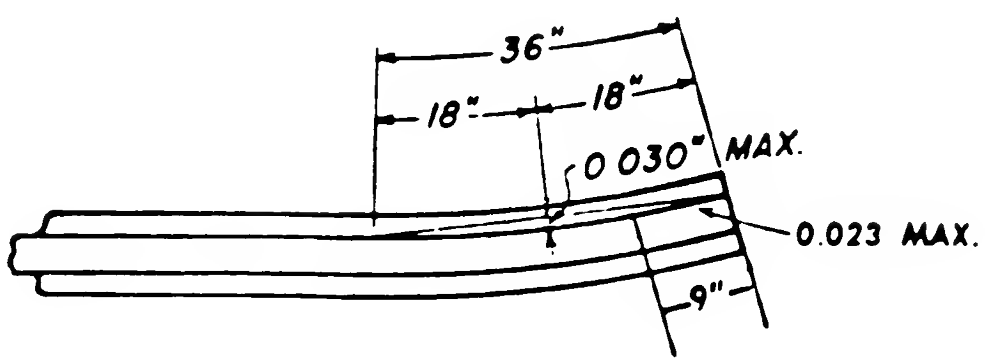


FIG. 3. Top View of Rail Lateral (Horizontal) Line Tolerance at Rail Ends per Section 13.5

13.6 When required, proof of compliance with 13.2 shall be determined by string (wire) lining, and a straightedge and taper gauge shall be used to determine rail end surface and line characteristics specified in 13.3, 13.4, and 13.5.

13.7 Rails shall be hot sawed, cold sawed, milled, abrasive wheel cut, or ground to length, as specified by purchaser, on purchase order, with a variation in end squareness of not more than 1/32 in. allowed. The method of end finishing rails shall be such that the rail end shall not be metallurgically or mechanically damaged.

14. Acceptance

14.1 To be accepted, the rails offered must fulfill all the requirements of these specifications.

14.2 Only A-rails produced on the purchaser's order will be accepted.

14.3 Rails accepted shall be shipped and invoiced based on the calculated weight per yard for the rail section.

15. Markings

15.1 High-strength rails shall be marked by either a metal plate permanently attached to the neutral axis, hot stamped, or in the brand which gives the manufacturer, type and/or method of treatment. Heat treated rail shall be paint-marked orange and alloy rail shall be paint-marked aluminum.

15.2 "A" rails shall be paint-marked yellow.

15.3 Rails except for those 78 ft. or 39 ft. shall be paint-marked green.

15.4 Individual rails shall be paint-marked only one color, according to the order listed above, or as agreed upon by purchaser and manufacturer.

15.5 Paint markings will appear on the top of the head at one end only, at least 3 ft. from the end.

15.6 All short length rails produced shall have the length identified in a manner acceptable to the purchaser and manufacturer on the top of the head approximately one foot from each end.

16. Loading

16.1 All rails shall be handled carefully to avoid damage and shall be loaded with the branding on all rails facing the same direction. Rails of different markings shall not be intermixed in loading, but shall be segregated and loaded head up. If there are not enough rails of one marking for a full car, smaller groups consisting of tiers of different markings as approved by the purchaser, may be loaded onto one car.

SUPPLEMENTARY REQUIREMENTS

The following supplementary requirements shall apply only when specified by the purchaser in the inquiry, order, and contract.

S1. End Hardening

S1.1 The drilled ends may be specified to be end hardened. When so specified, end hardening and chamfering shall be in accordance with S1.1.1 through S1.1.7.

S1.1.1 End-hardened rails may be hot stamped with letters "CH" in the web of the rail ahead of the heat number.

S1.1.2 Water shall not be used as a quenching medium except in oil-water or polymer-water emulsion process approved by the purchaser.

S1.1.3 Longitudinal and transverse sections showing the typical distribution of the hardness pattern produced by any proposed process shall, upon request of purchaser, be submitted for approval before production on the contract is started.

S1.1.4 The heat-affected zone defined as the region in which the hardness is above that of the parent metal shall cover the full width of the rail head and extend longitudinally a minimum of 1-½ in. from the end of the rail. The effective hardness zone ½ in. from the end of the rail shall be at least ¼ in. deep.

S1.1.5 The hardness measured at a spot on the centerline of the head ¼ in. to ½ in. from the end of the rail shall show a Brinell hardness number range of 341 to 401 when decarburized surface has been removed. A report of hardness determination representing the product shall be given to the purchaser or his representative.

S1.1.6 The manufacturer reserves the right to retreat any rails which fail to meet the required Brinell hardness number range.

S1.1.7 Chamfering rail ends shall be done in such a manner as will avoid formation of grinding cracks.

S2. Ultrasonic Testing

S2.1 The rail may be specified to be ultrasonically tested for internal imperfections and pipe by the purchaser or manufacturer.

S3. Calibration and Operation of Instruments

a. The instrument shall be standard ultrasonic testing equipment acceptable to purchaser.

b. Transducer or sensor shall be standard dual transducer of 5 MHz acceptable to purchaser.

c. Test block shall be of purchaser's choice with the following characteristics: Material 4340 AISI Steel/Nickel Plate, manufactured in accordance with ASTM E 428. Dimension of test block and flat bottom hole shall also be of purchaser's choice.

d. Calibration of instrument shall be performed every 30 minutes.

e. When search unit is properly coupled to test block or web of rail, a back reflection should appear at full maximum height on the Cathode Ray Tube "Graticule."

f. Couplant shall be distributed over the area to be examined and search unit moved over the entire area in vertical or horizontal sweeps. Any indication above the initial trace line between the impulse and the back reflection will be regarded as a flaw, inclusion or void and shall be reason for rejection.

APPENDIX 1

Inasmuch as the controlled cooling of rails has proved a successful method for the elimination of hydrogen, the following procedure is presented as one which will meet the requirements of Section 7.1.

1. All rails shall be cooled on the hot beds or runways until full transformation is accomplished and then charged immediately into the containers. In no case should the rail be charged at a temperature below 725°F.

2. The temperature of the rails before charging shall be determined at the head of the rail at least 12 in. from the end.

3. The cover shall be placed on the container immediately after completion of the charge and shall remain in place for at least 10 hours. After removal of raising of the lid of the container, no rail shall be removed until the temperature of the top layer of rails has fallen to 300°F or lower.

4. The temperature of an outside rail or between an outside rail and the adjacent rail in the bottom tier of the container, at a location not less than 12 in. nor more than 36 in. from the rail end, shall be recorded. This temperature shall be the control for judging rate of cooling.

5. The container shall be so protected and insulated that the control temperature shall not drop below 300°F in 7 hours for rails 100 lbs. per yd. in weight or heavier from the time the bottom tier is placed in the container and 5 hours for rails of less than 100 lbs. per yd. in weight. If this

cooling requirement is not met, the rails shall be considered control-cooled, provided that the temperature at a location not less than 12 in. from the end of a rail at approximately the center of the middle tier does not drop below 300°F in less than 15 hours.

6. The purchaser shall be furnished a complete record of the process for each container of rails.

SPECIFICATIONS FOR QUENCHED CARBON-STEEL JOINT BARS

(CHANGES TO SPECIFICATIONS SHOWN IN ITALICS)

A. Revise the title on Page 4-2-12 to read:

Specifications for Quenched Carbon-Steel Joint Bars *and Forged Compromise Joint Bars*

B. Revise Item 1, Page 4-2-12 to read:

1. *Scope*
These specifications cover heat treated, carbon-steel joint bars *and forged compromise joint bars* for general use in standard railroad tracks.

C. Revise Item 4, Page 4-2-12 to read:

4. *Heating and Quenching*
The bars shall be uniformly heated for punching, slotting, shaping, *forging* and subsequently quenched. *Maximum depth of decarbonization layer of forged bars shall not exceed 0.040 inches.*

D. Revise Item 11, Page 4-2-13 to read:

11. *Number of Tests*

(a) One tension test and one bend test shall be made from each lot of 1000 bars or fraction thereof, but not less than one test for each heat on each day on which bars are heated and quenched.

(b) If any test specimen shows defective machining or develops flaws it may be discarded and another specimen substituted.

(c) If the percentage of elongation of any tension test specimen is less than specified in Art. 8 and any part of the fracture is more than ¾ in. from the center of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest *of additional specimen* shall be allowed *as per Section 12.*

E. Revise Item 13, Page 4-2-14 to read:

13. *Workmanship*
The bars shall be smoothly rolled, or *forged,* true to template and shall accurately fit the rails for which they are intended *and shall provide a true alignment of the gage and running surfaces of the two rails being connected. (Head easement is recommended per Figure 8 of this chapter.)* The bars shall be either sheared or sawed to length, and the punching and slotting shall conform to the dimensions specified by the purchaser. A variation of plus or minus 1/32 in. from the specified size of holes, or plus or minus 1/16 in. from the specified location of holes, and of plus or minus 1/8 in. from the specified length of joint bar will be permitted. Any variation from a straight line in a vertical plane shall be such as will make the bars high in the center. The camber in either plane shall not exceed 1/32 in. in 24-in. bars and 1/16 in. in 36-in. bars.

F. Revise Item 15, Page 4-2-14 to read:

15. *Marking and Stamping*

The name or brand of the manufacturer, the section designation, and the year of the manufacture shall be *hot stamped on the side of each of the rolled bars.* A serial number representing the heat shall be hot stamped on the outside of the web of each bar, near one end. *Each compromise joint bar shall also have the rail sections shown at each end along with the word "gage" or "out" to indicate on which side of rail bar is to be used. (If the compromise joint bars are interchangeable, the words gage and out will be omitted.)*

COMMITTEE 5—TRACK

The Committee recommends that the following material on reverse curves should be added to Manual Chapter 5, Part 3, Page 5-3-12.1.

Minimum Tangent Lengths Required Between Reverse Curves With Spirals and Superelevation

The minimum tangent length between reverse curves with spirals and superelevation should not be less than the length of the longest car that is to traverse the curves.

Consideration should also be given to the chord length being used by the automatic lining equipment when establishing the minimum tangent length.

The Committee recommends the following addition to specifications for Track Construction — Manual Chapter 5, Part 4:

APPENDIX II

MINIMUM SPECIFICATIONS FOR INDUSTRIAL TRACK CONSTRUCTION

1. Industrial track will be considered as Track in yard territory and servicing either a light or heavy industry, with speed limit of 10 MPH.

2. Tie Requirements — use treated mixed #2 and #3 ties no less than 8′0″ long, and spaced at maximum of 24″ c.c.

3. Rail — new or relay rail with no more than ⅜″ head wear. No rail less than 90 #, with properly drilled holes, should be used. Joint batter not in excess of $\frac{3}{16}$″.

4. Tie Plates — new or relay tie plates must be used on all ties. All plates must have same cant.

5. Spikes — ⅝″ × 6″ new or used cut track spikes should be used. Two rail holding spikes will be installed per tie plate. One additional spike should be considered for use on curves of 5° or more.

6. No superelevation shall be used.

7. Maximum curvature of 12° should be used. If sharper curvature is required, approval of servicing railroad will be necessary.

8. Maximum recommended grade is two percent. If stepper grade is required, approval of

servicing railroad will be necessary.

9. Turnouts—use #7 or greater. If smaller turnout is required, approval of servicing railroad will be necessary.

A. Box anchor every third switch tie across.

B. Suggest use of appropriate switch point guard on turnout side switch point with heavy traffic.

C. Refer to AREA Portfolio of Trackwork Plans as to track duty requirements and construction details.

D. Suggest one additional spike per tie plate should be used on curve closure rails in turnout.

10. Rail Anchors — use 16 anchors per 39 feet of track; four nonconsecutive ties box-anchored per rail. Same tie shall be box-anchored across.

11. Rail Joints — use 4-hole, 24-inch or 6-hole, 36-inch joint bars fully bolted with lock washers. Refer to Table, Page 5-5-4.13 in AREA Manual, for proper expansion requirements when laying rail. The rail-joint bar contact surface should be lubricated upon installation.

12. Ballast — use AREA #5 size stone, ballast furnace slag or equivalent. Ballast depth should be a minimum of six inches from bottom of tie. Ballast section must remain level with top of tie between adjacent tracks to provide level walking area for trainmen or as required by local Commissions.

13. Necessary gaging shall be done at the time rail is laid and, unless otherwise provided, gage shall be 4″ 8-½″ between points ⅝″ below the top of rail on the two inside edges of the rails.

14. Crossings — use flange timbers on each side of rail. Flange ways should be sealed with bituminous material to insure water will not drain into track crossing. Rail joints should be avoided in crossing area.

15. Appliances such as derails, wheel stops and bumping posts will be installed as required by the serving railroad.

The Committee recommends the following complete revision of the tamping specifications shown on Manual page 5-5-8, Chapter 5, Part 5, Track Maintenance.

TAMPING

5.8.1 — Definitions

8.1.1

A "tamping tool" is any appartatus that is brought in direct contact with the ballast for the purpose of compacting it under the tie.

8.1.2

A "hand tamper" is any tamping tool that is manually inserted into the ballast. The tool may be activated manually or by a power source.

8.1.3

"Power tamper" refers to any tamping machine that inserts tamping tools through the use of mechanical, pneumatic, electrical, or hydraulic power systems.

8.1.4

"Spot tamping" is lifting and tamping short sections of track to restore it to proper surface.

8.1.5

"Out-of-face" tamping is lifting and tamping the entire track to restore it to a uniform surface and cross level. Lifts should be made and tamped in increments of two inches maximum.

8.1.5.1

"Skin lift" is out-of-face tamping of the track where the nominal raise is one inch or less.

5.8.2 — Tamping Tools

8.2.1

Tamping tools should have sufficient head and face area, based on manufacturer's specifications, to compact ballast under the tie and should be repaired or replaced when worn.

8.2.2

Tamping tools should be chosen on the basis of their durability, availability, type of ballast to be tamped, and the amount of ballast to be placed under the tie.

5.8.3 — Methods of Tamping

8.3.1

Tamping tools should be inserted simultaneously on opposite sides of the same tie to prevent the tie from cocking, to insure that the ballast under the tie is completely compacted and that the rail is firmly seated on the tie plate.

8.3.2

When using power tampers in tandem, the machines should be of the same type and have identical tamping heads to produce uniform compaction.

8.3.3

In all tamping, ties should be tamped from twelve inches inside of the rail to the end of the tie. Tamping should not be permitted at the center of the tie to avoid centerbound track.

Regardless of the kind of ballast or the kind of power tamper used, two tamping tools should always be worked opposite each other on the same tie.

The track should be raised to true surface and the ties tamped to a tight bearing against the raised rail. For spot tamping, tamping picks, ballast forks, ballast spades, shovels, tamping bars, or power tampers may be used.

After all tamping operations, the cribs must be properly filled in and the track finished in accordance with the standard ballast section.

The Committee recommends the following new track tool plans be included in Manual Chapter 5, Part 6: (Plans 36 through 40): 3-lb. and 5-lb. hot cutter chisels, long and short drift pins and a 3-inch welders flatter. Also new Plans A through D covering tool contour dimensions replacing the information now shown on Manual pages 5-6-32 through 5-6-35 and corrections to existing Plans 3, 13, 17, 19, 32 and 35. These corrections and revisions were the result of a review of specifications and plans for typographical errors, missprints and out-of-date nomenclature. Special areas of interest were tool contours, tool eye details and requirements for tool handles.

Copies of these proposed Manual changes are available from headquarters at the cost of $2.00 for this document.

The Committee recommends the following addition to Manual Chapter 5:

Part 7
Rail Anchors

SPECIFICATIONS FOR RAIL ANCHORS

1982
General

Scope

These specifications define the requirements for one piece drive on and spring type rail anchors, both new and remanufactured, which either may be applied manually or with standard anchor machines that readily can be adapted to the anchor being considered.

Function

Rail anchors transmit static and dynamic longitudinal forces in the rail to the cross tie. A sufficient number of anchors are required to hold the rail in a fixed position.

Design

Rail anchors shall be designed:

(1) To function during their service life without damage to the rail base due to longitudinal forces or when applied or due to vibratory action.

(2) To be able to resist longitudinal and torsional forces exerted by skewed ties without damage to the base of rail.

(3) To be capable of being applied tightly against the tie either manually or by machine.

(4) For easy application, removal and successive re-application without appreciable loss of holding power as required by the following slip test.

Bearing Area

Rail anchors shall have sufficient bearing area and depth to minimize the possibility of the anchor damaging or becoming embedded in the tie under pressure and to prevent the anchor from overriding the tie.

(These specifications have not addressed the subject of specific minimum tie bearing area requirements or anchor depth. The actual initial contact area of each manufacturer's product will vary because of the curved and irregular configuration of the vertical face of the anchor; therefore, there will be some anchor embedment before the load is uniformly distributed over the vertical face of the anchor. It is entirely possible that by the time the anchor embeds itself into the tie that a portion of the anchor will bear against the tie plate thus nullifying tie bearing area requirements based on the allowable compressive stress perpendicular to the grain for the tie species being used.)

Slip Test

Rail anchor slippage shall be determined by applying a gradually increasing load directly to the end of the rail; the anchor, applied perpendicular to the rail base, shall be resisted by a fixed metal block positioned ⅝ths inches below the base of the rail. In the interest of safety, the fixture should have clamps or other engagement which will prevent the rail and anchor from slipping off the fixed block while the load is being applied, but which will not restrain the rail from slipping through the anchor. The rail used in conducting the test shall meet AREA dimensional specification without any variation in the width and thickness of the base.

The rail shall be preloaded to 500 pounds to allow the anchor to take its initial set including anchor lean. The location of the anchor at the drive on or applied side of the rail base shall be marked or fixed by a dial gauge. The load shall be increased at a rate not to exceed 1.0 inch per minute or 10,000 pounds per minute until it reaches 5,000 pounds where it shall be held for three minutes before measuring slippage, which shall not exceed a total of $\frac{1}{16}$ inches; the 5,000 pound load shall be held for an additional three minutes during which time there shall be no further slippage.*

Upon satisfactory completion of each test, the anchor being tested shall be removed, reapplied at a different location on the rail base, and shall meet the foregoing criteria for a second and a third successive test.

Slip tests shall be made at least once for each purchase order; however, on large orders the purchaser may require that additional tests be made.

***NOTE:** Some purchasers require that the load be applied to the end of the rail in increments varying from 500 to 1,000 pounds and held for 5 to 10 seconds to enhance uniformity of testing.

Application Test

Rail anchor quality control shall be determined by subjecting one anchor from each heat to an application test. The test shall be repeated three successive times on the same anchor with no fractures allowed.

For drive on anchors the test shall require a 275 foot-pound impact on the hook end of the anchor placed upright over a 30 degree angle wedge.

For spring type anchors the test shall require a force of sufficient magnitude to spread the anchor, without breaking, to a permanent dimension which exceeds the width of the rail base plus .05 inches, on which it is to be applied, by 6 percent. This measurement shall be made between points, in the same plane parallel to the rail base, where the anchor normally comes in contact with the rail base.

Retests and Rejection

Should a sample fail the application or slip test, a retest consisting of two anchors randomly selected from the same production lot shall be made. If both of these samples meet the test requirements, the anchors from that lot shall be accepted.

If either retest sample fails the application or slip test, all of the anchors in the production lot shall be rejected. Following rejection these anchors may be scrapped or reheat-treated at the manufacturers' discretion. Any reheat-treated anchors subsequently shall be tested using the original criteria.

Dimensions

The manufacturer shall carry out dimensional checks on 2 of every 500 anchors produced per line to assure compliance with all dimensional requirements. If either of these two anchors fail to conform, the manufacturer must take immediate remedial action to reject all non-conforming anchors.

Workmanship

The finished rail anchors shall not be marred or deformed and shall be free of detrimental defects, such as, laps, cracks, seams and decarbonized or burned steel.

Identification

Each rail anchor shall be hot-stamped to show the rail section, weight and year of manufacture. Remanufactured anchors must be clearly and permanently identified.

Bagging

All anchors shall be bagged fifty (50) acceptable anchors per bag for shipment or storage. All anchors within each bag shall be for the same rail size and shall be of the same type of anchor. All correctly filled anchor bags shall have the open end of the bag closed completely and securely bound in such a manner as to insure that the bound end will not reopen during handling and shipping.

Tagging

Each bag shall have securely affixed to the outside an inspector's tag which shows the date of manufacture, type of anchor and the intended rail sizes covering the anchors contained in that bag.

Shipping Tag

Each load shall have securely affixed to not less than 10 percent of its bags tags which show consignee name and address, customer order number, manufacturer's order number, date of shipment and quantity of anchors in that shipment.

Inspection

When specified, the inspector representing the purchaser shall have free entry to the manufacturer's facilities and shall be provided, on a no charge basis, all reasonable facilities to satisfy him that the materials being furnished are in accordance with the purchaser's specifications. The plant manager shall be notified prior to the inspection so that the facilities, materials and product shall be available for inspection.

Acceptance

To be accepted, the rail anchors offered must fulfill all the requirements of these specifications.

COMMITTEE 6—BUILDINGS

The Committee recommends that the existing Part 1 of Chapter 6 of the Manual be deleted and replaced with a rewritten Part 1, Specifications and General Design Criteria for Railway Buildings, as summarized below:

The specifications portion of this section covers the recommended organization of specifications for railway buildings. The format recommended is that used by the Construction Specifications Institute (CSI). General guidelines for bidding requirements, contract forms, general conditions, special conditions, and technical specifications are covered.

The General Design Criteria portion of this section brings to the attention of the Architect, Engineer or Contractor involved in the design and construction of railway buildings problems that are unique to railway buildings or are rarely encountered in the design or construction of other buildings. This section should be used in conjunction with design criteria for specific railway facilities presented in other sections of this Manual, as well as with standard design practices, construction methods and material specifications.

Copies of the full text of this proposed Manual change are available from headquarters at the cost of $1.00 for this document.

The Committee recommends that a new Part 15 "Inspection of Railway Buildings" be included in Chapter 6. The report on this subject was previously printed in AREA Bulletin 680 in December, 1980. The only change in this report will be the addition of the Manual's decimal format.

COMMITTEE 8—CONCRETE STRUCTURES AND FOUNDATIONS

The Committee has rewritten Part 16 "Reinforced Concrete Box Culverts" and recommends that it replace existing Part 16. The revisions primarily reflect minor editorial changes, clarification of certain items and a minor change in Figure 16-1 as regards transverse load distribution. Part 16 has also been renumbered to conform to the rest of the Manual.

Copies of the proposed Manual changes are available from headquarters at the cost of $1.00 for this document.

The Committee recommends that the following new Part 24, Drilled Shaft Foundations, be included in Manual Chapter 8:

Part 24

Drilled Shaft Foundations

24.1 General

24.1.1 Scope

This specification covers the description and general aspects of design, installation, inspection and testing of drilled shafts, also frequently referred to as drilled caissons, drilled piers, or bored piles.

This specification is intended to serve as guidelines in developing specific designs and construction specifications on a project basis.

For the purpose of this specification, the minimum diameter of these units shall be 30 inches. Drilled shafts with smaller diameters have been constructed, but are not included in this specification.

This specification relates primarily to single drilled shafts. The design of drilled shaft groups is not included in this specification.

24.1.2 Purpose and Necessity

Drilled shafts are used to transmit loads through soils of poor bearing capacity into rock or soil formations having adequate bearing capacity. Generally, single drilled shafts have load capacities much larger than piling due to their larger size and capability of belling to increase the bearing area without enlarging either the footing or the drilled shaft.

The selection of foundation treatment for a given site should be determined by subsurface conditions, and by economic considerations as there is often a choice of several types of foundations for a structure.

24.1.3 Definitions (See Figure 1)

Drilled Shaft — A machine and/or hand excavated shaft, concrete filled, with or without steel reinforcing, for the purpose of transferring structural loads to bearing strata below the structure.

Protective Casing — Protective steel unit, usually cylindrical in shape lowered into the excavation to protect workmen and inspectors from collapse or cave-in of the side wall.

Bell or Underream — an enlargement at the bottom of the drilled shaft made by hand excavation or mechanical underreaming with drilling equipment for the purpose of spreading the load over a larger area.

Socket — a shaft of equal or smaller diameter extended into the bearing material.

Toe — vertical section at bottom of bell.

Permanent Casing — a steel cylinder that is installed for the purpose of excluding soil and water from the excavations. It is used as a form to contain concrete placed for the drilled shaft and remains in place.

Temporary Casing — a steel cylinder that is installed for the purpose of excluding soil and water from the excavations. It may also be used as a form for the shaft concrete but is withdrawn after the shaft is placed.

24.1.4 Design Loads

Loading for drilled shafts shall be the design loads from the supported structure without application of load factors used for Load Factor design procedure. Design loads shall include the following:

Primary Forces:
- Dead Load
- Live Load
- Centrifugal Force
- Earth Pressure
- Bouyancy
- Negative Skin Friction

Secondary Forces (Occasional):
- Wind and Other Lateral Forces
- Ice and Stream Flow
- Longitudinal Forces
- Seismic Forces

When drilled shaft foundations are designed for both primary and secondary forces, the allowable load on the drilled shafts may be increased by 25 percent, provided that the size or number of drilled shafts is not less than that required for primary forces alone. In soils where downward movements of surrounding soil relative to the drilled shaft are expected to occur, axial

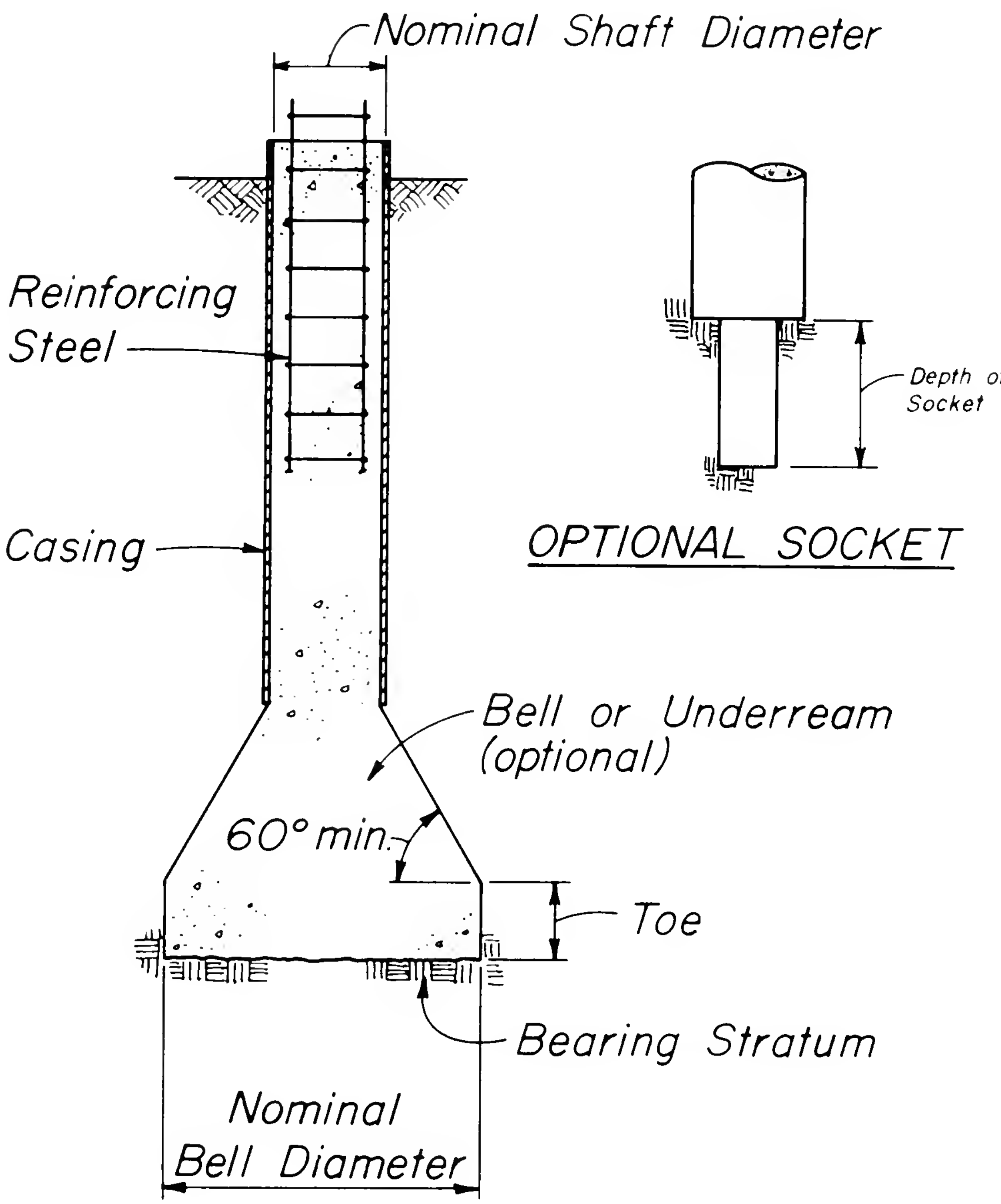


FIG. 1
DRILLED SHAFT

loads shall include negative skin forces, acting downward on the drilled shaft. Under special conditions swelling soils can produce upward forces, with fluctuation of the water table, which should also be considered in design.

24.2 Information Required

24.2.1 Field Survey

Sufficient information shall be furnished in the form of profile and cross sections to determine general design and structural requirements. The location of overhead and underground utilities, existing foundations, roads, tracks, or other structures shall be indicated. Records pertaining to high and low water levels and depth of scour shall be provided for stream crossings.

24.2.2 Subsurface Investigation

Foundation material shall be investigated as specified under Chapter 8, Part 22 Subsurface Investigation, in order to determine soil or rock properties, ground water elevations, and, any other pertinent conditions.

Reference is also made to Chapter 8, Part 4.3.1, for additional information.

24.3 Design

24.3.1 General

The design is divided into three basic parts: (1) the transfer of load from the drilled shaft to the rock and/or soil bearing strata, (2) the drilled shaft itself, (3) the connection between the supported structure and the drilled shaft.

24.3.2 The Transfer of Load from the Drilled Shaft to the Rock or Soil Bearing Strata

24.3.2.1 Drilled shafts transfer load to the bearing strata as follows:

a. An end bearing-type drilled shaft transfers essentially all of its load through weaker soils to a layer of soil or rock with adequate bearing capacity.

b. A combination end bearing and friction-type drilled shaft is a shaft in which some of the load is transferred into the stratum by skin friction and some by end bearing.

24.3.2.2 Lateral Loads and Moment: When the drilled shaft is subjected to lateral load and moment as well as axial load, the distribution of soil pressures and the variation of moments and shear in the shaft must be determined.[1]

24.3.2.3 Belled Shafts: Where the bearing strata has insufficient strength to support the load on the base of the shaft, the shaft bottom may be enlarged by belling or underreaming to reduce the pressure by distributing the load over a greater area. Belled shafts shall be used only where the soil/rock in which the bell is placed will not collapse due to the underreaming. Bells are normally unreinforced. The base diameter of the bell shall not exceed three times the shaft diameter and the sides shall not be less than 60 degrees from the horizontal. The toe height of bottom edge shall not be less than 3 inches for shafts from 30 inches to 42 inches in diameter and not less than 6 inches for shafts greater than 42 inches in diameter.

24.3.2.4 Allowable Capacity: Allowable values for end bearing, skin friction, and resistance to horizontal loads shall be determined by the Design Engineer in accordance with Part 3 this chapter.

24.3.2.5 Socketed Shafts in Rock: By socketing drilled shafts into rock, loads of greater magnitude can be transferred to the rock by a combination of end bearing and skin resistance. Skin resistance, similar to skin friction, is a resistance derived from adhesion or mechanical interlock at the periphery of the socket.

The socket should be proportioned so that the load is transferred to the rock by bearing on the end of the socket (drilled shaft) at the allowable bearing value of the rock, and the remainder of

1. Standard Specification for End Bearing Drilled Piers (ACI 336. 1-79) J, ACI, Sept. 1978.

the load is transferred by side resistance of the socket walls at the allowable adhesion between the concrete and rock. In accordance with this procedure, the required depth of rock socket or shaft embedment in rock materials may be determined by the following formula:

$$\text{Socket Depth, in inches} = \frac{R - f_b A_c}{f C_s}$$

R = design load (axial) at the base of drilled shaft, kips.

f_b = allowable bearing pressure on the rock, kips/sf.

f = adhesion value of the rock/concrete face, selected by the Design Engineer with approximate maximum value of 0.05 f_c, ksi.

A_c = cross-sectional area of concrete socket, sf.

C_s = circumference of socket, inches.

Where required by design assumption, the depth of the rock socket shall be adequate to obtain fixity.

High unit stresses are assigned to socketed shafts, therefore, it is essential that sockets be cleaned, inspected, and good concreting practice be maintained.

24.3.2.6 Shafts Under Water: Wherever practicable, the drilled shaft shall be designed to permit the placing of the concrete in the dry, and for visual inspection of the hole, the bearing strata, and the rock socket.

When it is impractical to dewater the excavation for rock-socketed shafts, the concrete may be placed under water by means of a tremie or pumped concrete and appropriate allowances made in the concrete mix design.

When concrete cannot be placed in the dry, and inspection is thus limited in nature, the Design Engineer shall reduce the allowable bearing stress appropriately.

Any free water in belled shafts shall be removed by pumping or bailing, and the bottom reinspected before placing concrete in the dry.

24.3.3 The Drilled Shaft

24.3.3.1 The drilled shaft is generally designed as a short column due to the lateral support provided by the soil/rock. In muck or water, slenderness effects of the column must be taken into consideration.

When the drilled shaft is subjected to moment and lateral forces at the connection to the supported structure, the shaft must be designed for bending and shear in addition to axial force. Moment and shear along the length of the shaft must be calculated, and adequate reinforcement provided.[1,2]

24.3.3.2 The shaft shall satisfy the design requirements of Part 2 of this Chapter, except that f_c shall equal 0.25 f'_c.

1. Reference design method: "Suggested Design and Construction Procedures for Pier Foundations" Reported by ACI Committee 336—Title No. ACI 69-42, J, ACI, Aug. 1972.

2. Reference design method: "Drilled Shaft Manual Vols. I and II, Reese, L.C., and Wright, S.L., U.S. Department of Transportation, Office of Research and Development, Implementation Package 77-21, July 1977.

24.3.4 Connection Between Supported Structure and Drilled Shaft

The connection between the drilled shaft and the supported structure (parts above the top of shaft) shall be capable of transferring the design loads, including direct load, shear and moment. This can be accomplished by the following means:

a. When the supported structure at the top of shaft is of concrete, the reinforcing steel cage shall be extended into the cap so that the load is transferred into the reinforcing steel of the drilled shaft by bond and into the concrete by compression.

b. When the cap section is a steel element, appropriate design shall be developed to transmit all loads, conforming to the requirements of Chapter 15, Part 1 or 2.

24.4 Material

24.4.1 Concrete

Unless otherwise stipulated in this specification, concrete shall be produced and placed in accordance with Part 1 of this chapter. Concrete shall have a minimum compressive strength of 3,000 psi in 28 days. Approved additives, such as set retarders, may be used to improve workability. Slump at time of placement shall be not less than 4 inches, and not more than 6 inches. If temporary casing is to be used, the slump should be not less than 5 inches, and a set retarder may be necessary.

24.4.2 Reinforcing Steel

Unless otherwise stipulated in this specification, any required reinforcing steel shall conform to the requirements of Part 1, this Chapter.

24.4.3 Steel Casing Material

If the steel casing is relied upon as a structural element, the steel casing material shall conform to the requirements of ASTM A252.

24.5 Construction

24.5.1 Contractor Qualifications

Drilled shafts shall be installed by the Owner with experienced personnel, or by a Contractor or Subcontractor who specializes in such work. Availability of all required special equipment, tools and experienced personnel are important items to be considered when determining Owner installation or selecting an installation contractor.

24.5.2 Shaft Excavation

When excavating a drilled shaft, earth walls shall be adequately and securely protected against cave-in, subsidence and/or displacement of surrounding earth, and for the exclusion of ground water by means of temporary or permanent steel casings.

Whenever personnel are required to enter the shaft, a protective casing shall be used and there shall be adequate provisions for fresh air, light and protection from falling objects and toxic gases. Operation of harmful gas-producing equipment in the shaft must be prohibited.

Rock grapples or special tools for removal of boulders or other obstructions must be readily available for use. Blasting will be permitted only upon obtaining written approval from the Engineer.

Inspection of the shaft base, and any socket, by a qualified inspector is highly recommended and should be omitted only with the approval of the Chief Engineer.

No shaft excavation shall be made within 15 feet of an uncased shaft filled with concrete that is less than one day old. The construction procedure used shall be approved by the Engineer in charge before commencing work.

24.5.3 Casing

Where called for, permanent steel casing shall be installed to the plan elevation or to the elevation designated by the Engineer in the field. When the top of the drilled shaft is below the surface of the ground, installation of additional larger diameter casing may be required to extend above the working level to minimize possibility of foreign materials or water entering the top of the shaft.

Casings shall be of adequate size and thickness to safely retain the adjacent earth materials and water from entering the shaft excavation, without exceeding allowable steel stresses, distortion, or collapse of the casing.

A protective casing is also to be provided, where required to serve as protection for personnel entering the shaft excavations not provided with casings as specified above. Casing size and thickness shall meet the requirements stated above. The outside diameter of the protective casing shall be as large as possible, yet small enough to be lowered and removed without damage to the sides of the shaft.

If conditions are such that casing withdrawal will cause dislocation of the reinforcing steel or permit sloughing soils to enter the shaft, a double casing should be used. By this method, the shaft is drilled oversize and a temporary casing installed. A light gage permanent inner casing the same size as the required shaft diameter is then installed. This inner casing shall be of sufficient strength to serve as a form for the concrete shaft but need not be designed for soil pressure. Concrete is then placed within the permanent inner casing. After the concrete has set, the annular space between the permanent casing and surrounding soil is filled with grout, lean concrete, sand or by other approved method and the temporary outer casing is withdrawn.

24.5.4 Bells or Underreams

Before belling, the Engineer shall determine that the formation encountered at the plan elevation is adequate. When shafts are required to be belled, the bells shall be formed either by hand or use of special belling equipment to the angle and slope called for on the drawings. The bottoms of bells shall be thoroughly cleaned of all loose materials, and inspected, before the concrete is placed.

24.5.5 Sockets

When sockets are required they shall be formed by machine or by hand to the proper size and depth called for in the plans. Sides and bottom of sockets must be thoroughly cleaned of all loose material since the bond of the concrete to the socket sides is used in design.

24.5.6 Tolerances

The center of the top of each shaft shall not vary from its design location by more than 1/24 of the shaft diameter, or 3 inches, whichever is less, and the shaft shall not be out of plumb by more than 1.5 percent of the length nor exceeding 12.5 percent of shaft diameter, whichever is less.

24.5.7 Dewatering

Suitable dewatering procedures shall be as agreed upon between the Engineer and Contractor as determined at such time as conditions warrant. Unless otherwise agreed, the shaft at the time of placement of steel reinforcing cage, if any, and concrete, shall be essentially free of standing water in excess of two inches deep.

24.5.8 Inspection

Immediately prior to placement of any required reinforcement or concrete, each shaft shall be thoroughly inspected as directed by the Engineer to ascertain that the shaft has been properly prepared, that the bearing material is compatible with design requirements, and whether additional investigation of the bottom is required. If conditions vary from the assumed conditions determined by the borings, additional investigation shall be conducted as directed by the Engineer.

24.5.9 Placing Steel

When reinforcing steel is specified, it shall be prefabricated and placed as a unit immediately prior to concrete operations. In order to minimize displacement of reinforcing steel cage when casing is pulled, the cage may be reinforced below the zone of significant bending moment by welding horizontal bands to the cage at about 5-foot intervals.

24.5.10 Placing Concrete

Dry Hole — Prevent segregation of concrete through use of tube, sectionalized pipe or other means to direct the free fall of concrete so that it does not strike the sides or reinforcement in the shaft.

Under Water — Utilize a tremie or pumped concrete in accordance with Chapter 8, Part 1, Article 1.14.10, and Part 24, Article 24.3.2.6.

Rodding or mechanical vibrating is necessary only for the top three feet of shaft. Any special requirements for concrete placement shall be approved by the Engineer.

24.5.11 Casing Removal

In situations where temporary casing is to be removed, the head of concrete inside the casing must be adequate to preclude infiltration of water and sluffage of the shaft face and sides.

Elasped time from beginning of concrete placement in cased portion of shaft, until extraction of casing is begun, shall not exceed one hour.

Extreme care shall be taken when a casing is removed to prevent subsidence of the surrounding ground if this condition is critical due to the presence of surrounding structures or utilities.

Elevation of top of the steel cage should be carefully checked before and after casing extraction. The top of the concrete shall not raise during extraction of the casing.

The exterior temporary casing, if a double-cased shaft, shall not be removed until three (3) days after the shaft is poured.

24.5.12 Continuity of Work

Drilled shaft construction work shall be planned so that all required operations proceed in a continuous manner until the shaft is complete. A precise time schedule agreement between the Contractor and the Engineer should be established. Provision shall be made for protecting the shaft and adjacent construction in case of unforseen interruptions. Such provisions shall be approved by the Engineer before drilling begins.

24.5.13 Records

An accurate record shall be kept of each drilled shaft as installed. The record shall show the top and bottom elevations, shaft and bell diameters, depths of test holes if required, date the shaft is excavated, inspection report of the bearing stratum, depth of water in excavation at time of placing steel and concrete, quantity of concrete placed compared with theoretical quantity, and any other pertinent data. Records shall be made and signed by both the project superintendent and inspector and distributed to proper authorities daily.

24.6 Testing

Materials used in construction of drilled shafts should be sampled and tested as specified elsewhere in Chapter 8.

Further testing of the shafts may be required by the Engineer in order to determine the quality of the concrete, by coring; or the bearing capacity of the shaft, by test loading. At least two (2) test cylinders shall be taken for each shaft.

The Committee recommends that the following new Part 40, ASTM Specification References, be added to Manual Chapter 8. This was originally old Part 24, which has been updated to conform with the contents of other parts in Chapter 8.

Part 40
ASTM Specification References

ASTM Specifications and Designations Referred to in this Chapter
1982

The following is a list of Standard cited in Chapter 8 which is supplied for the information of the Engineer designing and specifying materials in accordance with the Chapter.

The Standards are listed without year designations since the ASTM's may be modified annually. The various Parts of Chapter 8 are updated utilizing the latest edition of the ASTM at the time of updating of that Part. It is suggested that before specifying a Standard with a year designation, that the Engineer satisfy himself that there are no revisions to the Standards which would have a significant impact on the designs, materials or methods of test specified.

Designation	Specifications For or Method of Test For
A6	General Requirements for Rolled Steel Plates, Shapes, Sheet Piling and Bars for Structural Use
A36	Structural Steel
A53	Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless
A82	Cold Drawn Steel Wire for Concrete Reinforcement
A153	Zinc Coating (Hot-Dip) on Iron and Steel Hardware
A184	Fabricated Deformed Steel Bar Mats for Concrete Reinforcement
A185	Welded Steel Wire Fabric For Concrete Reinforcement
A242	High-Strength Low-Alloy Structural Steel

A252	Welded and Seamless Steel Pipe Piles
A276	Stainless and Heat-Resisting Steel Bars and Shapes
A307	Carbon Steel Externally Threaded Standard Fasteners
A325	High-Strength Bolts for Structural Steel Joints
A328	Steel Sheet Piling
A377	Gray Iron and Ductile Iron Pressure Pipe
A416	Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete
A421	Uncoated Stress-Relieved Wire for Prestressed Concrete
A441	High-Strength Low-Alloy Structural Manganese Vanadium Steel
A496	Deformed Steel Wire for Concrete Reinforcement
A497	Welded Deformed Steel Wire Fabric for Concrete Reinforcement
A500	Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
A501	Hot-Formed Welded and Seamless Carbon Steel Structural Tubing
A572	High-Strength Low-Alloy Columbian-Vanadium Steels of Structural Quality
A575	Merchant Quality Hot-Rolled Carbon Steel Bars
A588	High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick
A615	Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
A617	Axle-Steel Deformed and Plain Bars for Concrete Reinforcement
A663	Merchant Quality Hot-Rolled Carbon Steel Bars Subject to Mechanical Property Requirements
A675	Steel Bars and Bar Size Shapes, Carbon, Hot-Rolled, Special Quality Subject to Mechanical Property Requirements
A690	High-Strength Low-Alloy Steel H-Piles and Sheet Piling for Use in Marine Environments
A706	Low-Alloy Steel Deformed Bars for Concrete Reinforcement Structural Steel for Bridges
A722	Uncoated High-Strength Steel Bar for Prestressing Concrete
A767	Zinc-Coated (Galvanized) Bars for Concrete Reinforcement
A775	Epoxy Coated Reinforcing Bars
C29	Unit Weight and Voids in Aggregate
C31	Making and Curing Concrete Test Specimens in the Field
C33	Concrete Aggregates
C39	Compressive Strength of Cylindrical Concrete Specimens
C40	Organic Impurities in Fine Aggregates for Concrete
C42	Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C50	Sampling, Inspection, Packing and Marking of Lime and Limestone Products
C70	Surface Moisture in Fine Aggregate
C76	Reinforced Concrete Culvert, Storm Drain and Sewer Pipe
C87	Effect of Organic Impurities in Fine Aggregate on Strength of Mortar
C88	Soundness of Aggregates by Use of Sodium Sulphate or Magnesium Sulphate
C91	Masonry Cement
C94	Ready-Mixed Concrete
C109	Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm. Cube Specimens)
C114	Chemical Analysis of Hydraulic Cement
C117	Materials Finer than 75 um (No. 200) Sieve in Mineral Aggregates by Washing
C123	Lightweight Pieces in Aggregate
C125	Definition of Terms Relating to Concrete and Concrete Aggregates
C127	Specific Gravity and Absorption of Coarse Aggregate
C128	Specific Gravity and Absorption of Fine Aggregate

C131	Resistance to Abrasion of Small-Size Coarse Aggregate by Use of the Los Angeles Machine
C136	Sieve or Screen Analysis of Fine and Coarse Aggregates
C138	Unit Weight, Yield and Air Content (Gravimetric) of Concrete
C142	Clay Lumps and Friable Particles in Aggregates
C143	Slump of Portland Cement Concrete
C144	Aggregate for Masonry Mortar
C150	Portland Cement
C151	Autoclave Expansion of Portland Cement
C157	Length Change of Hardened Cement Mortar and Concrete
C173	Air Content of Freshly Mixed Concrete by the Volumetric Method
C185	Air Content of Hydraulic Cement Mortar
C190	Tensile Strength of Hydraulic Cement Mortars
C192	Making and Curing Concrete Test Specimens in the laboratory
C206	Finishing Hydrated Lime
C227	Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)
C231	Air Content of Freshly Mixed Concrete by the Pressure Method
C260	Air-Entraining Admixtures for Concrete
C289	Potential Reactivity of Aggregate (Chemical Method)
C295	Petrographic Examination of Aggregates for Concrete (Recommended Practice)
C309	Liquid Membrane-Forming Compounds for Curing Concrete
C330	Lightweight Aggregates for Structural Concrete
C342	Potential Volume Change of Cement-Aggregate Combinations
C360	Ball Penetration in Fresh Portland Cement Concrete
C443	Joints for Circular Concrete Sewer and Culvert Pipe Using Rubber Gaskets
C478	Precast Reinforced Concrete Manhole Sections
C494	Chemical Admixtures for Concrete
C506	Reinforced Concrete Arch Culvert, Storm Drain, and Sewer Pipe
C507	Reinforced Concrete Elliptical Culvert, Storm Drain and Sewer Pipe
C535	Resistance to Degradation of Large Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine
C567	Unit Weight of Structural Lightweight Concrete
C595	Blended Hydraulic Cements
C618	Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
C641	Staining Materials in Lightweight Concrete Aggregates
C655	Reinforced Concrete D-Load Culvert, Storm Drain, and Sewer Pipe
C666	Resistance of Concrete to Rapid Freezing and Thawing
C778	Standard Sand
C789	Precast Reinforced Concrete Box Sections for Culverts, Storm Drains, and Sewers
C850	Precast Reinforced Concrete Box Sections for Culverts, Storm Drains, and Sewers with Less Than 2 ft. of Cover Subjected to Highway Loadings
D25	Round Timber Piles
D75	Aggregates, Sampling
D245	Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber
D1143	Piles Under Axial Compressive Load
E11	Wire-Cloth Sieves for Testing Purposes

COMMITTEE 10—CONCRETE TIES

The Committee recommends the following new Manual material for Chapter 10, Articles 1.2.4 through 1.2.5 (to replace existing Articles 1.2.4 through 1.2.5.2); Article 1.9.1.3 (to replace existing Article 1.9.1.3); and an entirely new Article 1.9.1.15.

These changes will require Article 1.9.1.2 Sequence of Design Tests (tie "2") part (b) to be changed as follows:

1.9.1.2 (b) Fastening Uplift Test (described in Art. 1.9.1.10 parts (a) (b) shall be performed on one rail seat.

Article 1.9.1.10 Fastening Uplift Test will be changed to read as follows:

(a) An 18-to 20-inch piece of the proper section of rail shall be secured to one rail seat using a complete rail fastening assembly, including pads, bolts, clips and associated hardware, as recommended by the manufacturer of the rail fastening system. In accordance with the loading diagram in Figure V, an incremental load shall be applied to the rail. The load P (measured load plus unsupported tie weight plus frame weight) at which separation of the rail from pad or pad from rail seat (whichever occurs first) shall be recorded.* The load shall then be completely released.

(b) A load of 1.5P not to exceed 10 kips shall then be applied. The inserts shall not pull out or loosen in the concrete and no component of fastening system shall fracture nor shall the rail be released.

The above mentioned Articles 1.2.4 through 1.2.5, Article 1.9.1.3, and Article 1.9.1.15 will read as follows:

1.2.4 TIE PADS

Tie pads shall be used between the rail and concrete ties to minimize tie abrasion of the rail seat area, and reduce impact and vibration effects on the track structure.

1.2.4.1 Requirements

Tie pads shall be of dimensions and materials suitable for use with the fastener and track structure components. Pad material shall provide the required chemical and physical properties to resist effects of environment exposure and traffic loads, and to satisfy requirements of the test specified in Section 1.9.

1.2.4.2 Material Tests

For elastomeric pads, supplier shall submit results of industry standard tests concerning the following properties:**

(a) Compression set at high temperature (ASTM D395).

(b) Compression set at low temperature (ASTM D1229).

(c) Tensile strength and elongation before and after heat aging (ASTM D412 and ASTM D573).

(d) Hardness (ASTM D2240).

(e) Abrasion resistance (ASTM D2228).

*"P" shall not exceed 10 kips.

**Tests for non-elastomeric pads are currently under study

(f) Resistance to fluids such as water, acids, alkali, petroleum oils, and synthetic lubricants (ASTM D471).

(g) Resistance to ozone (ASTM D518).

Test conditions and acceptance maximum/minimum ranges should be established by the engineer with consideration given to environment exposure, traffic loads, speed, and other related factors.

1.2.4.3 Tie Pad Tests

From a lot not less than 10 pads produced, three pads shall be selected at random for laboratory testing. Tie pad tests shall be conducted using a tie block, as described in Article 1.9.1.15, following the Sequence of Design Tests specified in Article 1.9.1.3.

1.2.5 INSULATION

Insulation shall be used where necessary to prevent interference with signal systems and deterioration of the fastening system through electrical leakage. Insulation may be provided by insulators placed at appropriate locations in the fastening assembly or by other acceptable means.

Insulators shall be of dimensions and materials suitable for use with the fastener components. Insulator material shall provide the required chemical and physical properties to resist effects of environment exposure and traffic loads, and to satisfy the requirements of the tests specified in Section 1.9.

1.9.1.3 Sequence of Design Tests (Tie Block)

The sequence of design performance tests using the tie block shall be as follows:

(a) Tie Pad Test* (described in Art 1.9.1.15).

(b) Fastening Uplift Test Part A (described in Art. 1.9.1.10).

(c) Fastening Longitudinal Restraint Test (described in Art. 1.9.1.12).

(d) Fastening Repeated Load Test (described in Art. 1.9.1.11).

(e) Fastening Longitudinal Restraint Test (described in Art. 1.9.1.12).

(f) Fastening Uplift Test Part A (described in Art. 1.9.1.10).

(g) Fastening Laterial Restraint Test (described in Art. 1.9.1.13).

(h) Tie Pad Test (described in Art. 1.9.1.15).

1.9.1.15 Tie Pad Test

(a) The tie pad shall be loaded vertically using a rail section in a manner similar to its use in the fastening system.

(b) A cyclic load varying from 4,000 to 30,000 lb shall be applied continuously at a rate of 4 to 6 cycles per second for a total of 1,000 cycles.

(c) A static load shall be applied, at a rate between 3,000 and 6,000 lb/min., in increments of 1,000 lb up to a maximum of 50,000 lb. For each load increment, vertical pad deflection shall be measured to the nearest 0.0001 in. and recorded. The recorded values for vertical load versus deflection shall be plotted on a graph. Spring rate, as determined by the slope of the line connecting the points representing pad deflections at 24,000 and 44,000 lb, shall be calculated.

(d) Load shall be released and pad deflection recorded 10 seconds after load removal.

*Test shall be conducted on three pads. The two pads providing highest and lowest spring rate values shall be used for tests (b) through (h)

(e) The requirements of this test will have been met, if

i. Pad returns to within 0.002 in. of its original position 10 seconds after load removal.
ii. Spring rate values determined from both pad tests, conducted as part of the design performance tests specified in Article 1.9.1.3, do not vary by more than 25%.
iii. Spring rate values determined from initial tests (a) conducted on the three test pads, as part of the design performance tests specified in Article 1.9.1.3, do not vary by more than 25%.
iv. Spring rate values determined from final tests (h) conducted on the two test pads, as part of the design performance tests specified in Article 1.9.1.3, do not vary by more than 25%.

COMMITTEE 13—ENVIRONMENTAL ENGINEERING

The Committee recommends that the existing Part 1 of Manual Chapter 13 be deleted and replaced with a rewritten Part 1, Water Pollution Control, as summarized below:

This section of the Manual deals with handling of pollution problems due to sanitary and industrial wastewaters. The section was essentially rewritten from scratch as understanding of the problems and advances in technology had rendered the previous edition, about ten years old, outdated.

The revised section is divided into three subsections:

1.1 Sanitary Sewage Treatment and Disposal

1.2 Industrial Wastewater Treatment and Disposal

1.3 Flow Monitoring, Sampling, Sample Preservation, Instrumentation and Analysis

A lengthy bibliography is also included.

The first subsection deals with sanitary or domestic wastes. This subsection received the least change, however some paragraphs dealing with antiquated or environmentally unacceptable technologies have been deleted. Certain design details that vary and are specific to various geographical areas have been deleted. Sources for correct detail selection criteria are included in replacement.

The subsection on industrial waste received the greatest change. Extensive emphasis was placed on proper identification of waste sources (shop drainage, fuel area runnoff, etc.) and source control where possible. Considerable experience was available to the authors relative to wastewater collection and treatment. Discussion of methodologies which have proven successful, as well as those which failed, is included. Several common design mistakes are discussed to spare the inexperienced engineer the same grief. Treatment technologies and various levels of treatment are discussed relative to certain discharge limitation levels.

The subsection on monitoring has been updated to reflect new, acceptable testing methods as well as experience in the mechanics and equipment for sampling and flow measurement.

In general the revisions to Part 1 were directed at creating a useful, practical document, primarily to direct the engineer inexperienced in this field.

Copies of the full text of this proposed Manual change are available from headquarters at the cost of $3.00 for this document.

COMMITTEE 15—STEEL STRUCTURES

The Committee offers the following *text* material for inclusion in Part 6 of Manual Chapter 15.

6.1.2 Abbreviations

(a) The following abbreviations are used herein:

AAR	Association of American Railroads
AFBMA	Anti-Friction Bearing Manufacturers Association
AGMA	American Gear Manufacturers Association
AISI	American Iron and Steel Institute
ANSI	American National Standards Institute
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
IEEE	Institute of Electrical and Electronics Engineers
IPCEA	Insulated Power Cable Engineers Association
JIC	Joint Industrial Council
NEC	National Electrical Code
NEMA	National Electrical Manufacturers Association
NFPA	National Fluid Power Association
SAE	Society of Automotive Engineers

The National Fire Protection Association shall be referenced herein by full name only.

6.1.4

(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 JIC Hydraulic Standards, Section H1.10.

6.1.5

(a) Where the machinery design is prepared by the Contractor, he shall furnish complete calculations for all parts of the machinery. The calculations shall include the operating shaft torques for all pump drive motors and drive engines, hydraulic motors and rotary actuators and rod forces for hyudraulic cylinders and intensifiers along with hydraulic system pressures. Calculations shall be for the following conditions:

6.1.12

(c) The Contractor shall also furnish six bound copies of a similar booklet for the hydraulic equipment, which shall conform to the requirements of JIC Hydraulic Standards, Section H1.12.4.

6.1.14

(a) Class 13, Hydraulic Equipment — All hydraulic equipment, including hydraulic fluid and portable filtration units used during reservoir filling, necessary to provide the operating system specified, whether directly or indirectly associated with the system, shall be considered as a part of this work.

Hydraulic equipment shall also include mechanical and electrical equipment normally mounted on the power unit such as electric motors, couplings, coupling guards, and accessories such as pressure, temperature and fluid level switches, immersion heaters and gauges.

6.4.8 Hydraulic Systems and Components

6.4.8.1 Allowable System Pressures

(a) The hydraulic system shall be designed and hydraulic components proportioned such that the maximum allowable system pressures shall not exceed the following, except as otherwise permitted by prior written approval of the Company.

Normal operation:	1000 psi
Operation against maximum specified loads:	2000 psi
Holding against maximum specified wind loads:	3000 psi

(b) Normal operation shall be defined as operation against Condition A loads specified in Art. 6.3.6. Operation against maximum specified loads shall be defined as operation against Conditions B & C loads specified in Art. 6.3.6. Holding against maximum specified wind loads shall be defined as holding the movable span in the fully open position, static condition, against the loads specified in Art. 6.3.6(e).

6.4.8.2 Pressure Ratings for Hydraulic Components

(a) Minimum working pressure ratings for hydraulic components shall be as follows, except as otherwise permitted by prior written approval of the Company.

Pipe, tubing and their fittings:	3000 psi
Flexible hose and hose fittings:	
For pressure lines	5000 psi
For drain lines	2000 psi
Cylinders, pumps, valves and all other components:	3000 psi

(b) Working pressure rating shall be defined as the maximum allowable continuous operating pressure for the component. For pipe, tubing, flexible hose and fittings the working pressure ratings are equal to the burst pressure rating divided by a minimum factor of safety of 4. For cylinders the working pressure rating shall be equal to the NFPA theoretical static failure pressure rating as required by Art. 6.5.37.11 divided by a minimum factor of safety of 3.33. For pumps, valves and other components the working pressure rating is equal to the maximum allowable peak (intermittent) pressure rating divided by a minimum factor of safety of 1.5.

(c) The minimum factors of safety designated in paragraph (b) apply to systems having light to moderate operating shock loads during operation resulting in short duration peak system pressures no greater than two times the allowable maximum operating pressure against Conditions B or C loads, whichever is greater. For systems having higher shock load pressures, the factors of safety shall be increased proportionally.

6.5.37 Hydraulic Systems

6.5.37.1 Drawings

(a) Design drawings and specifications shall conform to JIC Hydraulic Standards, Sections H1.10 and H1.11, and will be furnished by the Company or, if stated in the invitation for bids, by the Contractor.

(b) Design drawing originals shall be on reproducible material, preferably mylar.

(c) As-built changes made by the Contractor shall be recorded by the Contractor in accordance with JIC Hydraulic Standards, Section H1.12.3.

6.5.37.2 Identification

(a) All hydraulic components shall be identified in accordance with JIC Hydraulic Standards, Section H1.16, including those located within manifolds, mounting plates, pads or fittings.

6.5.37.3 Accessibility

(a) All hydraulic components shall be mounted, located and arranged to be readily accessible for adjustment and maintenance.

(b) Hydraulic components shall be located such that the adjustment and maintenance of one component does not disturb the adjustment or maintenance of another.

(c) Accessibility of piping, fittings and manifolds shall conform with the requirements of JIC Hydraulic Standards, Section H11.21.

6.5.37.4 Safety

(a) Requirements for safety shall conform with the JIC Hydraulic Standards, Section H13.

6.5.37.5 Hydraulic System Controls

6.5.37.5.1 Methods of Operator Control

(a) Hydraulic system controls shall be designed to permit the bridge operator to control, from the control station, the rate and direction of fluid flow for span movement and the operation of auxiliary equipment such as rail locks, span locks, wedges, barriers and other devices associated with the movement of the span. Controls shall be of the type that will automatically maintain constant fluid flow, within 5% of maximum flow during full load change, without operator assistance regardless of normal operating pressure fluctuations, except during periods of acceleration and deceleration. Requirements for hydraulic system controls shall conform with the JIC Hydraulic Standards, Section H3.

(b) Methods of operating the hydraulic system shall be classified as manual, semi-automatic or automatic control, as follows:

(1) Manual control shall be defined as any system in which the operator must manually control the rate of fluid flow for span acceleration and deceleration in addition to the initiation of each of the several major interlocked functions in sequence.

(2) Semi-automatic control shall be defined as any system where the fluid flow automatically increases from zero to normal volume and back to zero again for span acceleration and deceleration by the single operation of a push-button or hand lever. However, the operator must initiate each of the several major interlocked functions in sequence.

(3) Automatic control shall be defined as any system where the operator can actuate the several major interlocked functions in sequence and the hydraulic system fluid flow automatically increases from zero to normal volume and back to zero again for span acceleration and deceleration, all by the operation of a single pushbutton or hand lever.

Requirements for the sequencing of bridge functions, span speed control and interlocking shall be in conformance with Art. 6.7.5.

(c) Flows produced by fixed displacement pumps shall preferably be controlled by varying the speed of the pump drive motors. If pressure compensated flow controls are provided to control fixed displacement pump flow, the hydraulic system shall be designed to minimize heat build-up.

(d) Variable displacement pump flows shall be directly controlled by manual stroking, or if remotely controlled, preferably by closed loop servo control systems.

(e) Closed loop servo control systems shall be analyzed by the manufacturer of the servo control components to verify that the control system will perform as required. The servo component manufacturer shall furnish all necessary instructions on how to trim (adjust) and maintain the servo control system.

6.5.37.5.2 Control Stations

6.5.37.5.2.1 Location and General Requirements

(a) The Operator's control station shall be located for either direct (valve station) or remote (control console) operation of the hydraulic system. Direct operation shall be defined as hydraulic system control from the power unit or separate valve stand, by the use of manually operated directional or flow control valves, or the manual stroking of variable displacement pumps. Remote control shall be defined as hydraulic system control from a control console. Remote control shall be accomplished with push-buttons or hand levers (joysticks) to operate solenoid controlled directional and flow control valves for fixed displacement pumps, or electrically operated servo valves or gear-motors for the stroking of variable displacement pumps.

(b) Indicating lights, gauges and other warning devices shall be provided at the control stations to monitor and protect the hydraulic system from damage due to low pressure, high pressure, low fluid level, high fluid level, low temperature, high temperature and pump servo valve malfunction. Requirements for pressure gauges shall conform to Art. 6.5.37.20.

6.5.37.5.2.2 Valve Stations

(a) Nameplates shall be provided for each control in accordance with the JIC Hydraulic Standards, Section H13.2.

(b) Control valves for manual control shall be located as shown on the drawings or, if not shown, at a comfortable working height and oriented in such a way that water and railroad traffic can be readily observed by the operator.

(c) Control valve handles for manual span operation shall be located for right handed operation by the operator. If separate control valves are provided for manual brake, lock or wedge operation, they shall be located for left handed operation by the operator.

(d) When valves are to be controlled by the operator's right and left hands simultaneously they shall preferably be located no more than 3 feet apart and in no case more than 4 feet apart.

(e) Pressure gauges shall be provided to monitor hydraulic system pressures and shall be located where they can be easily observed by the operator during bridge operation.

6.5.37.5.2.3 Control Consoles

(a) Requirements for control consoles, instruments, position indicators and indicating lights shall conform with Art. 6.7.5.

6.5.37.5.3 Pressure Controls

(a) Adjustable pressure control valves shall be provided in the hydraulic system to maintain desired pressure levels and to protect equipment from damage due to excessive operating and static pressures.

6.5.37.5.4 Shock and Surge Suppression

(a) The hydraulic system and its controls shall be designed to minimize shock loads from pressure surges during system operation.

(b) Automatic or pre-programmed acceleration and deceleration shall preferably be provided for span operation. Span movement controls shall preferably be designed such that if the operator tries to change direction of the span while it is moving, the span will decelerate smoothly to standstill and then smoothly accelerate to the same set speed in reverse.

(c) Directional control valves and blocking valves shall be equipped with adjustable pilot control chokes for shock and surge pressure control if the velocity of the hydraulic fluid in the piping exceeds 20 feet per second.

(d) Flexible hose may be used between fixed components to help control shock and surge pressures. When used for this purpose flexible hose and hose fittings shall have a minimum factor of safety as defined in Art. 6.4.8.2.

(e) Deceleration valves and/or accumulators shall be used in hydraulic systems with moderate to severe shock and surge pressures.

(f) Piping clamps shall have cushioned inserts to reduce vibration and noise and help to absorb shock in the piping system.

6.5.37.5.5 Temperature Control

(a) Any unusual high or low temperature that affects hydraulic equipment operation shall be noted in the Special Provisions.

(b) Reservoir hydraulic fluid temperature shall not be permitted to fall below 45°F during periods of hydraulic system inactivity. Immersion and/or unit heaters controlled by automatic thermostats shall be provided where ambient temperatures fall below 45°F.

(c) Reservoir hydraulic fluid temperature shall not be permitted to rise above 140°F during periods of hydraulic system operation. Reservoirs shall be sized large enough to dissipate heat and be located to have an adequate amount of free air circulation. If reservoir sizing and free air circulation will not control heat build-up, then heat exchangers shall be provided. Requirements for heat exchanges shall conform with JIC Hydraulic Standards, Section H10.

(d) Hydraulic systems originally not requiring heat exchangers but using fixed displacement pumps and relief valves, or pressure compensated flow control valves, for purposes of pump unloading, shall have provisions at the power units for the addition of heat exchangers at a later time, after the system has been installed at the bridge, if heat build-up during operation becomes excessive.

6.5.37.5.6 Synchronization of Actuators

(a) Flow dividing devices or other means shall be provided in the hydraulic system for the synchronous operation of end lifting devices or other equipment which is not mechanically connected but which must be synchronized for proper operation.

6.5.37.6 Hydraulic Power Units

6.5.37.6.1 General Requirements

(a) Hydraulic power units shall conform to NFPA Standard T3.16.3M Requirements for Non-Integral Industrial Fluid Power Hydraulic Power Units.

(b) The Contractor shall make assembly drawings, drawn to scale, of the hydraulic power unit. Each component of the hydraulic power unit, including piping, shall be identified. Nameplates shall preferably be shown on the assembly drawings at their actual locations on the hydraulic power unit. If nameplates cannot be shown on their actual locations a keyed nameplate list shall be provided on the power unit drawing. The Contractor shall submit the assembly drawings to the Company for approval.

(c) Work shall not begin on the power unit until the shop drawings have been approved. Shipment of the hydraulic power unit to the bridge site for installation will not be permitted until it has been demonstrated to the Company that the unit has the ability to perform as specified.

Power units shall be tested in conformance with Art. 6.5.37.25.2.

(d) Where the bridge operator is to operate the hydraulic system directly at the power unit, the overall height and location of the power unit shall not interfere with visibility of navigation or trains.

(e) Requirements for piping, fittings and manifolds for power units shall conform with Art. 6.5.37.10.

(f) Requirements for couplings to connect pumps to drive motors shall conform with Art. 6.5.37.16.

(g) Hydraulic fluid shall be filtered as it is placed into the reservoirs, both during original reservoir filling and during the addition of make-up fluid. The fluid shall be filtered while being pumped from its original containers using portable filtration units. The degree of filtration shall be equal to 10 microns or the same as that used during normal hydraulic system operation, whichever is finer.

6.5.37.6.2 Pumps

(a) Requirements for pumps shall conform with the JIC Hydraulic Standards, Section H5 and the NFPA Standards.

(b) Pumps shall be positive displacement of either the variable or fixed displacement type. Pumps shall be equipped with integral or add-on relief valves to prevent damage to pump and hydraulic system from high pressure. Relief valves shall not discharge into pump intake ports.

(c) Piston type or gear type pumps shall be used in hydraulic systems where maximum operating pressures, as defined in Art. 6.4.8, exceed 2000 psi.

(d) Where noise control is an important consideration, such as when the hydraulic power unit is to be located in the bridge house, piston pumps shall be used.

6.5.37.6.3 Pump Actuators

(a) Requirements for servo valve pump actuators shall conform with the JIC Hydraulic Standards, Section H2.7.

(b) Servo valve pump actuators shall be of the type which automatically return the pump to the neutral or zero pumping position in the event of pump control system malfunction, loss of electrical power or loss of hydraulic control pressure. Valves capable of by-passing 100% of pump volume shall be provided in the hydraulic circuit to by-pass fluid flow in the event of loss of servo control and the servo actuator does not return to neutral.

(c) Pump actuators shall have provisions for manual operation of the pump.

(d) The use of pneumatically operated actuators for pump control shall not be permitted.

6.5.37.6.4 Fluid Reservoirs

(a) Fluid reservoirs shall conform to NFPA Standard T3.16.2M. Nonintegral Industrial Fluid Power Hydraulic Reservoirs, except where noted herein.

(b) Reservoirs shall be of heavy-duty welded steel construction. They shall be structurally rigid to resist warpage and damage from the mounting of equipment on the reservoir top, handling during shipping and erection at the bridge site.

(c) Reservoirs interior and exterior surfaces shall not be galvanized. Painting of interior surfaces shall not be permitted. Interior surfaces shall be coated with a vapor-phase rust inhibitor specially formulated to prevent rust. Rust inhibitor shall be added to the hydraulic fluid by the hydraulic power unit manufacturer prior to testing and shipment from the factory.

(d) Bladder-type breathers to prevent the mixing of outside air and reservoir air shall be provided for fluid reservoirs located in environments having airborne contaminants such as dust, chemicals and condensing water vapor which can damage the hydraulic system.

(e) Reservoirs shall have drains which permit a complete fluid change without disconnecting any hydraulic components.

(f) Reservoirs equipped with large removable covers shall have separate filler openings to permit the adding of fluid to the reservoir without removal of the cover. Requirements for all filler openings shall conform with NFPA Standard T3.16.2M.

(g) Reservoirs shall either be equipped with accessories as specified in Art. 6.5.37.6.6 or shall have provisions for future installation of these accessories. The reservoirs shall be constructed to permit the addition of accessories, without disturbing existing equipment, after the hydraulic system has been put into operation.

6.5.37.6.5 Electric Motors

(a) The general requirements for electric motors, control and overload components shall conform with Art.6.7.5 except where noted herein.

(b) Electric motors used for driving of hydraulic pumps, except as required in (d), shall preferably be squirrel cage induction types. Motors shall be 1800 or 1200 rpm, TEFC or TENV, types with embedded winding temperature-sensitive devices, as specified in the contract documents. Motors shall have grease-lubricated antifriction shaft bearings and shall be equipped with lubrication fittings.

(c) Electric motors for the driving of variable displacement pumps or fixed displacement pumps utilizing pressure compensated flow control valves shall be squirrel cage type, NEMA design B and shall have manual across-the-line or simple reversing starters.

(d) Electric motors for the driving of fixed displacement pumps, where there is no provision for controlling the rate of fluid flow, shall be AC wound rotor induction motors or direct current motors. Speed controls for these motors shall be provided in conformance with Art.6.7.5.

6.5.37.6.6 Accessories

(a) Hydraulic power units shall be equipped with the required accessories to protect the hydraulic equipment from damage and ensure the safety of maintenance and operating personnel. Accessories shall include but not be limited to such items as gauges and transmitters for pressure, temperature and fluid level monitoring, immersion heaters, fluid conditioning filters and magnets, heat exchangers and air dryers or coalescing filters for reservoir vents.

(b) Requirements for pressure gauges shall conform to Art. 6.5.37.20.

(c) Requirements for filters shall conform to Art. 6.5.37.18.

(d) Requirements for heat exchangers shall conform with JIC Hydraulic Standards, Section H10.

(e) Immersion heaters shall be electric resistance type and controlled so as not to cause deterioration of the hydraulic fluid from overheating. Preferably dry-well type immersion heaters shall be used. Steam or hot-water coils shall not be used for immersion heating.

(f) Cold-water coils shall not be used for reservoir oil cooling.

6.5.37.7 Internal Combustion Engine Pump Drives

(a) Internal combustion engines, for the driving of pumps, shall be permitted only for emergency operation, or at locations where suitable electric power cannot be provided.

(b) Requirements for internal combustion engines shall conform to Art. 6.7.4.

(c) Manual or electrically operated clutches shall be provided for the coupling of engines to pumps. Clutches shall be of the type that engage gradually and smoothly, and will slip during equipment overloads, to prevent damage to pumps or engines.

(d) Electrically operated clutches shall be normally disengaged and shall have electric power applied to engage and remain engaged. Electric clutches shall have provisions for manual operation.

(e) Requirements for pumps, reservoirs, pump actuators, accessories and piping shall conform to Art. 6.5.37.6 and 6.5.37.10.

6.5.37.8 Valve Stands

(a) Valve stands for the mounting of manifolds, valves and gauges may be either integral parts of the hydraulic power units, or separate floor-mounted units. Separate floor-mounted valve stands shall be provided when the overall size or weight of the hydraulic power units with integral valve stands is so great that shipping, erection or maintenance may be difficult.

(b) Valve stands shall be of heavy duty construction, rigidly constructed to resist deflection and warpage during shipping, erection or operation of the system.

(c) Where the bridge operator's control station is located at the valve stand, the overall height and location of the valve stand shall not interfere with visibility of navigation or trains.

6.5.37.9 Valves

(a) Requirements for valves shall conform with the JIC Hydraulic Standards, Section H8.

(b) Adjustable valves shall be equipped either with protective caps, or with locking nuts on the adjusting screws, to prevent unintentional misadjustment.

(c) Directional control valves and blocking valves shall be provided with adjustable pilot control chokes to increase valve opening and closing time, for shock and surge pressure control.

(d) Flow dividing valves used for actuator synchronization shall be of the type that will always permit flow to all actuators simultaneously, regardless of the magnitude of pressure differential between the actuators being loaded.

6.5.37.10 Piping, Fittings and Manifolds

6.5.37.10.1 General Requirements

(a) Piping shall include all pipe, tubing and flexible hose. Requirements for piping, fittings, manifolds and the piping system in general shall conform with the JIC Hydraulic Standards, Section H11, except as otherwise noted herein.

(b) Piping, fittings and manifolds shall be made of carbon steel or stainless steel. The materials used shall be consistent with the pressures and environmental conditions to which the hydraulic system will be subjected. Steel fittings shall be used with steel piping, and stainless steel fittings shall be used with stainless steel piping. Use of fittings which are softer than the piping shall not be permitted. Piping, fittings and manifolds shall not be galvanized.

(c) Fittings used for piping connections shall be of the type to permit rapid assembly and disassembly of all components. Fittings shall also permit repeated disassembly and reassembly of a connection without loss of sealing quality or strength.

(d) Pipe shall preferably have welded flange fittings. Use of threaded pipe fittings in pressure lines above 200 psi shall not be permitted without prior written approval of the Company.

(e) Tubing shall have flared, flareless or welded flange fittings. Use of flared fittings shall be limited to tubing of 1-½-inch nominal outside diameter or smaller. Flareless fittings may be used for tubing sizes up through 2 inch nominal outside diameter. Welded flange fittings shall be used for tubing of greater than 2 inch nominal outside diameter.

(f) Fluid velocity in pressure and return line piping shall not exceed 15 feet per second and pump suction line velocity shall not exceed 5 feet per second unless approved in writing by the Company by the time shop drawings are reviewed.

(g) The Contractor shall make piping layouts and assembly drawings for the hydraulic system. These drawings shall clearly indicate the type and spacing of piping supports. The Contractor shall submit the drawings, and they must be approved by the Company, before field erection will be permitted. Support spacing and type shall conform to JIC standards.

(h) Test ports shall be provided to bleed the system of air, and to check system pressure at control valves as well as other locations where a pressure governing component is not so equipped.

(i) Flexible hose shall be provided to connect the hydraulic power unit to the rigid piping system. Where separate valve stands are provided, flexible hose shall be used to connect the valve stands to the hydraulic power unit and to the rigid piping system.

(j) Piping shall be connected to hydraulic component ports by means of SAE Straight Thread O-ring fittings for piping sizes up to ⅞ inch nominal outside diameter, and by means of SAE split flange fittings for larger size piping. The use of tapered pipe thread fittings to connect piping to components will not be permitted.

6.5.37.10.2 Pipe

(a) Pipe shall be seamless with plain ends. Use of threaded pipe ends will not be permitted, without prior written approval of the Company.

(b) Carbon steel pipe shall conform to the following ASTM specifications:

A 53 — Type S, Grade B
A 106 — Grade B
A 714 — Type S

(c) Stainless steel pipe shall conform to ASTM specification A 312 — Grades TP304 or TP316.

The foregoing materials are of the minimum quality that shall be used for pipe. Other materials of greater strength and durability may be specified by the Company.

6.5.37.10.3 Pipe Fittings

(a) Welded flange fittings shall be SAE 4-bolt minimum flanges, utilizing a captive O-ring pressure sealing system. Socket weld flanges shall preferably be used. Use of threaded flanges will not be permitted without prior written approval of the Company. Carbon steel flanges shall be manufactured from low carbon steel to facilitate welding. Stainless steel flanges shall be type 304 or 316 and be suitable for welding. Flange connecting bolts shall be hardened and have sufficient strength for the working pressure rating of the flanges. Stainless steel bolts shall be used with stainless steel flanges. A lockwasher shall be used at every bolt.

(b) Threaded fittings and threaded flange fittings, used for field connections and field erected piping systems, when approved for use above 200 psi, by the Company, shall have Dryseal Pipe Threads to permit pressure-tight joints without the use of pipe sealing compound, or PTFE sealant tape.

6.5.37.10.4 Tubing

(a) Tubing shall be seamless, have a low carbon content and be annealed to facilitate bending and flaring. Tubing to be used with flareless fittings shall have a maximum hardness of

65 Rockwell B.

(b) Carbon steel tubing shall conform to the following ASTM specifications:

A 519 — Grades 1010, 1020 and 4130
A 423 — Grades 1 and 2
A 669

(c) Stainless steel tubing shall conform to ASTM specification A 269 — Grades TP304 or TP316.

The foregoing materials are of the minimum quality that shall be used for tubing. Other materials of the strength and durablility required may be specified by the Company.

6.5.37.10.5 Tube Fittings

(a) Flared fittings shall have a 37° angle of flare and conform to SAE specifications.

(b) Flareless fittings shall conform to SAE specifications and be of the type that bites into the outside surface of the tubing when the fitting assembly is tightened.

(c) Welded flange fittings shall be SAE 4-bolt flanges utilizing a captive O-ring pressure sealing system. Socket weld flanges shall preferably be used. Use of threaded flanges will not be permitted without prior written approval of the Company. Carbon steel flanges shall be manufactured from low carbon steel to facilitate welding. Stainless steel flanges shall be grade 304 or 316 and be suitable for welding. Flange connecting bolts shall be hardened and have sufficient strength for the working pressure rating of the flanges. Stainless steel bolts shall be used with stainless steel flanges. A lockwasher shall be used at every bolt.

Threaded flange fittings, when approved for use by the Company, shall have Dryseal Pipe Threads to permit pressure-tight joints without the use of pipe sealing compound or sealant tape.

6.5.37.10.6 Flexible Hose and Fittings

(a) Only extra-high or high pressure hose conforming to SAE specifications and having the working pressure ratings specified in Art. 6.4.8.2(a) shall be used. Hose shall be seamless, oil and weather resistant and have steel wire reinforcement.

(b) Hose fittings shall be made of steel and be of the pressed-on (non-reusable) type conforming to SAE specifications. Hose fittings shall have either 37° SAE flare or flange style ends for connection to other hydraulic components. Flange head style fittings shall use SAE split flanges with hardened bolts and O-ring sealing. Threaded fittings may only be used for connection to threaded drain ports.

6.5.37.10.7 Special Fittings

(a) Special fittings of the swivel, rotating or self-sealing type shall not be used without prior written approval of the Company.

(b) Quick-disconnect type fittings shall not be used except for the temporary connection of portable gauges to test ports and the temporary connection of hand or air operated hydraulic pumps for emergency or maintenance operation of the hydraulic system.

6.5.37.10.8 Manifolds

(a) Requirements for manifolds shall conform with the JIC Hydraulic Standards, Section H11.

6.5.37.11 Cylinders

(a) Requirements for cylinders shall conform with NFPA standards. Cylinders shall have a minimum NFPA theoretical static failure pressure rating of 11 (10,000 psi), as defined by NFPA

Standard T3.6.5M.

(b) Cylinders shall have engraved permanent nameplates which are securely attached to the head of the cylinder. The nameplates shall clearly indicate the manufacturer, model number, cylinder bore, rod diameter, stroke length, NFPA theoretical static failure pressure rating symbol, and all features which are non-standard.

(c) Protective flexible rod boots shall be provided for all cylinders that are oriented such that the rods are normally extended.

(d) Piston rod seal assemblies shall be replaceable without cylinder disassembly.

(e) The use of rotating type or telescoping cylinders shall not be permitted without prior written approval of the Company.

6.5.37.12 Intensifiers

(a) The use of intensifiers or pressure boosters shall not be permitted without prior written approval of the Company. Intensifiers shall only be used to assist in the operation of auxiliary equipment such as locks, lifting devices, wedges, brakes and barriers. Intensifiers shall only be used for holding or clamping purposes and shall not be operated continuously as a pump.

(b) The maximum output pressure from an intensifier shall be 3000 psi and components subjected to the boosted operating pressure shall be designed to withstand the pressure with a factor of safety as defined in Art. 6.4.8. Relief valves shall be provided in the boosted circuits to protect equipment and personnel.

(c) The use of air-air or air-oil intensifiers shall not be permitted.

6.5.37.13 Fluid Motors

(a) Requirements for fluid motors shall conform with the JIC Hydraulic Standards, Section H4 except as otherwise noted herein.

(b) Fluid motors shall be of the fixed displacement type. Speed control of the motors shall be accomplished by controlling the volume of fluid to the motors.

(c) Gear type fluid motors shall be of the hydraulically balanced type.

(d) Where hydraulic systems use fluid motors in which the operating pressure, as defined in Art. 6.4.8, exceeds 2000 psi, only piston type fluid motors shall be used.

(e) High speed fluid motors shall be coupled to driven equipment in a manner that eliminates overhung loads on the fluid motor's shaft bearings. The magnitude of overhung loads on low speed, high torque (LSHT) fluid motor shafts shall perferably be limited to provide a minimum shaft bearing B-10 lift rating of 20,000 hours. (B-10 life shall be as defined by the AFBMA and shall be the time for which 90% of a group of identical bearings will survive under the given loading condition).

(f) Requirements for couplings to connect fluid motors to other equipment, for purposes of transmitting fluid motor operating torques, shall comply with Art. 6.5.37.16.

6.5.37.14 Rotary Actuators

(a) Rotary actuators are devices which produce output torque over a limited range of rotation, usually less than 360°. Actuators shall self-lock when the flow of pressurized fluid to the actuator is stopped or operating pressure is lost due to line leakage or breakage. Provision for manual operation of actuators shall be provided. Vane type rotary actuators shall be hydraulically pressure balanced.

(b) Actuators shall have keyed output shafts and be connected to driven equipment with couplings conforming to Art. 6.5.37.16.

(c) Cylinder type rotary actuators having internal chain and sprocket mechanisms shall have automatic chain tensioning devices incorporated into the actuators.

(d) Actuators shall be coupled to driven equipment in a manner that eliminates overhung and thrust loads on the actuator shaft bearings.

6.5.37.15 Self-Contained Hydraulic Actuators

(a) A self-contained hydraulic actuator unit shall consist of a heavy duty cylinder or other type of actuator, electric motor, pump, reservoir and control valving. Units shall be completely closed systems, requiring no external piping to supply or remove hydraulic fluid.

(b) Self-contained hydraulic actuators shall not be used for span operation. Such actuators may be used only to operate auxiliary equipment such as locks, lifting devices, wedges and barriers.

(c) Cylinders shall conform to NFPA standards.

(d) Electric motors shall conform to the general requirements of Art. 6.7.5.

(e) Protective rod boots shall be provided for cylinder rods which are normally extended.

6.5.37.16 Couplings

(a) Requirements for couplings, connecting pumps to drive motors and connecting fluid motors or rotary actuators to speed reducers or other equipment, shall conform to NFPA Standard T3.16.3M except as otherwise noted herein.

(b) Coupling guards shall be provided that conform to the requirements of NFPA Standard T3.16.3M.

(c) The use of belts for coupling purposes will not be permitted.

(d) Rigid coupling of equipment will not be permitted without prior written approval of the Company.

(e) Chain casings shall be provided for chain-type couplings. Casings shall be designed to seal in lubrication, and protect sprocket teeth and chains from abrasives.

(f) The use of shock resistant couplings, with non-metal torque transmitting components, will be permitted only where the coupling design is such that normal operating torques can be transmitted by the coupling in the event of non-metal component failure.

6.5.37.17 Brakes

(a) Machinery brakes or counterbalance valving, for span driving cylinders, shall be provided to hold the span stationary against unbalanced loads and the wind pressures specified in Arts. 6.3.6(e) and (f).

(b) If the hydraulic system does not provide sufficient braking to stop the span in 10 seconds or less, dynamic brakes shall be provided.

(c) Machinery brakes shall have the capacities as specified in Art. 6.3.9. Dynamic brake capacity shall be the same as specified for motor brakes in Art. 6.3.9. Electrically operated brakes shall conform to the requirements of Art. 6.7.5.

(d) Spans normally left in the open position shall also be provided with locking devices to hold the span stationary at the fully open position, against the wind loads specified in Art. 6.3.5(b)2.

6.5.37.18 Filtration and Fluid Conditioning

(a) Requirements for filtration of hydraulic fluid shall conform with the JIC Hydraulic Standards, Section H7.

(b) Full flow filtration shall be provided.

(c) Filters, including pump intake strainers, shall be equipped with an indicator to show when the filter needs servicing.

(d) The degree and quality of filtration shall be as recommended by the manufacturer of the hydraulic components. The Beta 10 rating system as defined by ANSI standard B93.31 shall be used to determine filter performance. Filtration performance shall never be allowed to deteriorate, at time of filter change, below a Beta 10 rating of 15 for servo controlled hydraulic systems, and a Beta 10 rating of 5 for non-servo controlled hydraulic systems.

(e) Filter flow capacity ratings shall be as recommended by the pump manufacturer. As a general guide, such capacity shall be equal to at least 10% of high pressure pump capacity for hydrostatic (non-differential) drives, at least 30% of high pressure pump capacity for a normal industrial type differential system, and 100% of high pressure pump capacity for differential system operating in a contaminated atmosphere.

(f) Bypass valves shall be provided on filters to limit the differential pressure across the filter elements. Bypass valves shall be sized for the maximum flow that can be expected through the filter without excessive differential pressure. Non-bypass type filtration shall be used only where required by the hydraulic equipment manufacturer, and shall be equipped with warning devices to provide remote indication at the operator's station of an impending clogged condition.

6.5.37.19 Accumulators

(a) Requirements for accumulators shall conform with the JIC Hydraulic Standards, Section H9 and the NFPA standards.

(b) Gas accumulators shall be charged with an inert gas such as dry nitrogen or heliun. The use of oxygen, air or other active gases will not be permitted for accumulator charging.

(c) Clamps or straps used for accumulator mounting shall not restrict thermal expansions, or distort the shell of the accumulator.

6.5.37.20 Pressure Gauges

(a) Gauges shall be of durable construction. Dial faces shall be clearly calibrated for pressure ranges 50% and beyond the maximum design operating pressures of the hydraulic system. Gauges shall be accurate and permit continuous monitoring. They shall have a minimum diameter of four inches, and preferably six inches. Shut-off valves shall be provided at each gauge.

(b) Portable gauges shall be provided for maintenance and adjustment of the hydraulic system. The pressure ranges shall cover all possible values that will be needed for the system. One gauge shall be provided for each pressure range such that the test pressure will be within the mid-half of the total pressure range of the gauge.

(c) Connections for portable gauges shall be of the quick-disconnect type. Test ports in the hydraulic system shall be equipped with removable, protective caps, secured by chains to the component. Shut-off valves shall be provided at each test port.

6.5.37.21 Hydraulic Fluids

(a) Hydraulic fluid shall be suitable for the operating pressure, temperature and lubrication requirements of the system. The selection of the hydraulic fluid shall be based on performance data or actual experience in other heavy duty hydraulic systems subjected to similar operating pressures and temperatures and having similar hydraulic equipment. The fluid shall be that recommended by the pump manufacturer and shall be compatible with all hydraulic components and seals.

(b) Hydraulic fluid shall be either petroleum based oil type, or oil-water emulsion type fire resistant fluid which is compatible with the same type of seals used with petroleum based oils. Straight synthetic, high water content fluids, synthetic blends or water-glycol mixtures shall not be permitted without prior written approval of the Company.

(c) Hydraulic fluid shall have the correct viscosity range for the operating requirements of the hydraulic system; shall have a high enough viscosity index to resist changes in viscosity due to anticipated temperature ranges, prevent wear on working parts, resist foaming, oxidation and the formation of sludges; shall retain original properties in use; and shall have a long service life and protect parts against rust.

6.5.37.22 Seals and Sealing Devices

(a) Requirements for seals and sealing devices shall conform with the JIC Hydraulic Standards, Section H12.

6.5.37.23 Workmanship

6.5.37.23.1 Piping Systems

(a) Piping runs shall be as short and free of bends as possible. At least one bend shall be provided in pipe runs where thermal expansion and contraction may be a problem.

(b) Piping bends shall be of good quality without excessive flattening or creasing. Minimum bend radius shall be 3 times the inside diameter of the pipe. Each leg of a piping bend shall have a length of not less than 10 pipe outside diameters.

(c) Tubing flares shall preferably be formed with roller type flaring tools.

(d) Bending and flaring shall be done with suitable portable equipment at the bridge site.

(e) Bolted flange connections shall be evenly assembled by the use of feeler gauges and torque wrenches to ensure equal bolt tightening. O-rings shall be lubricated before flanged connections are assembled.

(f) The use of pipe compound or sealant tape to facilitate the assembly of threaded fittings will not be permitted.

6.5.37.24 Field Painting

(a) Nameplates on all hydraulic components shall not be painted. Protective tape shall be placed over all nameplates prior to field painting, and subsequently removed.

(b) The final coat of paint, for field erected piping systems, shall preferably have a color such that hydraulic fluid leakage will be easily observed.

(c) Flexible hoses and hose guards shall not be painted.

6.5.37.25 Testing

6.5.37.25.1 Components

(a) Pumps and fluid motors shall be tested by the manufacturer before hydraulic power units are assembled, and catalog rating certification shall be provided to the Company. Tests for pumps and fluid motors shall be conducted for 15 minutes continuously, at a minimum test pressure equal to the maximum peak or intermittent pressure rating of the component.

(b) Pumps shall be checked during testing for external leakage, charge pump pressure and flow (where charge pumps are provided), and main pump pressure and flow. Integral relief valves shall be set at 3000 psi maximum, and checked for proper operation.

(c) Fluid motors shall be checked during testing for external leakage, pressure and flow.

(d) Cylinders shall be tested by the manufacturer before shipment to the bridge site. Testing shall include a 30 minute static pressure test at a minimum pressure of 4000 psi. The leakage rate past the piston during the static pressure test shall be no greater than 5 cubic inches per minute for span driving cylinders. Certified test data for span driving cylinders and catalog rating certification for all other cylinders shall be provided to the Company.

6.5.37.25.2 Power Units

(a) Assembled power units shall be shop tested for proper operation, and certified test data submitted to the Company for approval before shipment to the bridge site.

(b) Power units shall be shop tested at full drive motor speed under conditions of maximum design pressure at minimum fluid flow, and reduced pressure at maximum fluid flow. The maximum pressure test shall be conducted for one hour continuously.

(c) During all tests, the power units shall be checked for fluid leakage, excessive fluid temperature, proper relief valve operation and proper operation of charge pumps.

(d) Pump controls shall be tested for correct speed regulation, response time and direction of rotation.

6.5.37.25.3 Shipment

(a) Power units, valve stands and cylinder assemblies shall be shipped fully assembled to the bridge site and installed at their final positions. Pumps, motors and couplings shall be checked for proper alignment and realigned if necessary. Disassembly of power units, valve stands and cylinder assemblies will not be permitted for shipment, storage or during installation.

(b) Hydraulic equipment fluid ports shall be securely sealed prior to shipment and shall remain sealed until final assembly of the hydraulic system. Seals shall not be removed until just before the connection of components.

6.5.37.25.4 Final Tests at Bridge

(a) After final installation, but before connection to the piping system or valve stands, power units shall be checked for correct rotation of drive motors and pumps.

(b) Reservoirs shall be filled with fluid to the correct level. Portable filtration units shall be used during reservoir filling in conformance with Art. 6.5.37.6.1(f).

(c) When the entire installation is completed, the span, including all accessories, shall be operated by the Contractor through not less than three complete cycles using normal power, prime movers, and controls; and through at least two cycles using auxiliary or emergency power, prime movers, and controls. These tests shall be repeated for alternate operating modes if provided.

(d) During these tests, equipment shall be inspected for external fluid leakage, and to determine whether all features are in proper working order and adjustment, and whether they meet the requirements of the drawings and specifications.

(e) Portable pressure gauges shall be used at all test stations of the hydraulic system, including the power unit.

(f) During all tests, the level of the hydraulic fluid in the reservoir shall be closely monitored. Proper fluid level shall be maintained at all times to prevent pump cavitation. Air shall be bled from the hydraulic system and make-up fluid added to the reservoir as required, using portable filtration units in conformance with Art. 6.5.37.6.1

(g) In the event tests show that any features are defective or inadequate, or function improperly, the Contractor shall make all necessary corrections, adjustments, or replacements at his own expense.

(h) When all the components are in proper working order and adjustment, the pressure readings taken at each test station shall be recorded, and provided to the Company.

(i) After completion of final tests hydraulic fluid shall be removed, properly discarded, replaced with new fluid, and air bled from the entire hydraulic system. New fluid shall be added using portable filtration units in conformance with Art. 6.5.37.6.1.

In lieu of fluid replacement, the Contractor may take fluid samples from each reservoir for analysis by the fluid supplier. The fluid shall be changed if sample contamination levels are greater than Class 3, as defined by specification SAE ARP-598. New fluid, where required, shall be added using portable filtration units in conformance with Art. 6.5.37.6.1

(j) After completion of final hydraulic testing, and either fluid replacement or the continued use of fluid which has passed contamination level testing, filter elements shall be replaced and strainers and magnets cleaned.

The Committee recommends the following additions to Chapter 15, Part 8 of the Manual:

8.6 GUIDELINES FOR EVALUATING FIRE DAMAGED STEEL RAILWAY BRIDGES

8.6.1 Introduction

(a) The evaluation of a railway bridge after a fire has one primary goal, and that is to determine the ability of the structure to continue to carry railroad loading. To do this, an examination of what has happened to the steel during a fire must be made. The reaction of steel to a fire can be broken down into two areas. The first area consists of temporary changes that occur while the steel is at elevated temperatures, and the second area is made up of permanent changes. It is these permanent changes that are of the most concern.

8.6.2 Types of Fires

(a) A railway bridge may be subject to three basic types of fires: a deck fire, consisting of ties and timber guardrail; a brush fire or fire from an adjacent structure; or a cargo fire. A deck fire or brush fire is usually of short duration, and is unlikely to cause serious damage to the bridge except possibly for the stringers supporting the deck. A fire in an adjacent structure or a cargo fire are potentially the most hazardous because of the possible high temperatures developed for a long period.

8.6.3 Temperature Effects

(a) The temporary changes in steel due to elevated temperatures include decreased strength, decreased modulus of elasticity and increased coefficient of expansion. These temporary effects can, however, combine into the permanent effect of distortion. Table 8.6.1 lists the properties of structural carbon steel relative to temperature:

TABLE 8.6.1 — PROPERTIES OF STRUCTURAL CARBON STEEL RELATED TO TEMPERATURE

Temperature	Yield Strength	Tensile Strength	Modulus of Elasticity	Coefficient of Expansion
Atmospheric	100%	100%	100%	100%
400°F	90%	100%	95%	106%
800	60%	85%	85%	117%
1050	50%	50%	80%	125%
1300	20%	15%	70%	132%
1600	10%	10%	50%	—

These effects, while temporary, can cause the bridge to collapse during the fire.

(b) Included in the permanent effects on steel due to elevated temperatures are decrease in strength, decrease in elasticity and distortion.

(1) The strength lost by a member due to heating above approximately 1100°F is only that extra strength imparted to it during rolling. The basic strength of the structural steel is not lost. Finally, if steel is heated to very high temperatures for long enough periods, the surface of the steel will oxidize. This is evidenced by a heavy scaling and pitting and indicates a loss of strength in the steel. The amount of time necessary to oxidize steel is dependent on temperature, with less time being needed at the higher levels of temperature. At 1200°F, 6 to 7 hours are required. At 2000°F only approximately one half an hour is needed. While the temperature of a fire may be quite high, it does not necessarily follow that the steel reached that temperature. It takes approximately one hour per inch thickness of steel for thorough heating.

(2) The modulus of elasticity of steel will decrease from a 100% value at atmospheric temperature to as low as 50% of that modulus at 1600°F, as indicated by Table 8.6.1.

(3) Distortion occurs in two basic forms: buckling of small light members and warping or buckling of large heavy members. A small light member such as bracing is constrained at both ends. Heating such a member produces compressive stresses in the member. The associated loss of strength allows buckling to occur, and the decrease in elasticity makes this buckling permanent. A large heavy member such as a girder cannot be uniformly heated. This uneven heating causes warping or buckling. This same effect can cause distortion during welding and is also the principle behind flame straightening or cambering. Distortion can occur after temperatures as low as 450°F and is, therefore, not a precise indication of the maximum temperature reached by the steel during a fire. When steel is heated above its transformation temperature (1300° to 1550°F) and quickly cooled, it will lose some of its ductility. If steel is heated above approximately 1100°F and cooled slowly, it will lose part of its as-rolled strength. It is these last two changes, which are not readily discernible, that cause the most concern. However, the quick rate of cooling needed to harden steel is virtually impossible to achieve at a fire site. The use of water from a fire hose is usually insufficient to cause hardening, but may increase the distortion experienced by a member.

8.6.4 High Strength Steels

(a) The comments of Art. 8.6.3 on the effect of temperature do not necessarily apply to high strength steels which have achieved their strength through heat treatment. Such steels must be given individual consideration and may require laboratory study.

8.6.5 Fasteners

(a) Fasteners, either rivets or bolts, will begin to lose their clamping force at approximately 600°F. They should be carefully inspected and if there is any indication that the fasteners have been affected by the fire, they should be replaced. This is normally a simple routine, and one that should be followed to ensure long term liability.

(b) Fasteners in connections of distorted members may also be subjected to high tensile forces which can result in popped rivet heads or broken bolts. This condition may occur away from the fire zone.

8.6.6 Evaluation of bridge

(a) To evaluate a railway bridge after a fire the following data are useful:

1. Maximum temperature reached by the steel.
2. Length of time maximum temperature was maintained.
3. Information on the physical condition of the steel

a. distortions

b. scaling, pitting, etc.

c. hardness

4. Laboratory test results of specimens taken from structure.

(b) Items 1 and 2 are usually not available, or if available are only estimates. The information in Item 3 can be obtained by an examination of the steel in the field and is the most important. Item 4 is often impractical considering the time involved, and while it may eliminate many doubts, it is usually unnecessary.

(c) The most obvious physical change resulting from fire is distortion. While distortion may be grounds for rejection, it is not necessarily an indication of a lessening of the strength of the steel. If a member can be straightened economically, it usually can be reused. A member with only minor distortions may be usable without repair.

(d) Scale will start to form on steel at approximately 900°F. From approximately 900° to 1000°F the resulting scale will be red in color. A black scale will form from approximately 1200° to 1400°F. If oxidation has occurred a heavy dark gray scale will form. It is only this heavy scaling that indicates damage to the steel and is cause for rejection. Such heavy scaling will be accompanied by pitting and loss of section, and is easily identifiable.

8.6.7 Conclusion

(a) In conclusion, if the steel is undistorted, or can be economically straightened, it is generally safe for reuse. The only exceptions are members showing evidence of heavy oxidation, which is usually recognizable, and fasteners. Fasteners will start losing a substantial amount of their clamping force at approximately 600°F and should be thoroughly investigated. Generally speaking, it is advisable to replace any fasteners showing evidence of having been affected by the fire.

(b) This conclusion is drawn for the simpler types of railroad bridge structures. If a complex structure having interacting framing, continuity, and/or indeterminate characteristics is involved, the possibility of high locked in tensile stress in restrained elements that have yielded and cooled must be considered. This condition can result in brittle fracture when subsequently exposed to cold weather conditions. It can also subject connections and fasteners to large forces.

8.6.8 References

(a) Dill, F.H.; "Structural Steel After A Fire", Proceedings of the American Institute of Steel Construction Engineering Conference, 1960.

(b) Lie, T.T. and Stanzak, W.W.; "Structural Steel and Fire — More Realistic Analysis", Engineering Journal/American Institute of Steel Construction, 1976.

(c) Schirmer Engineering Corporation, Rolf Jensen & Associates, Inc.; "Fire Experience and Exposure in Fixed-Guideway Transit Systems", Prepared for American Iron and Steel Institute, December, 1980.

(d) Siegal, L.G.; "The Severity of Fires in Steel-Frame Buildings", American Institute of Steel Construction Engineering Journal, October, 1967.

(e) Uppal, A.S.; "Evaluation and Repair of Fire Damaged Steel Bridges", Special Project Report No. 2, American Railway Bridge and Building Association, Chicago, Illinois 1981.

(f) Wakiyama, Kozo and Tatsumi, Akio; "Residual Force in High Strength Bolts Subjected to Heat", Technology Reports of the Osaka University, Vol. 29, No. 1488, Osaka, Japan, 1979.

COMMITTEE 28—CLEARANCES

The Committee recommends the following revisions to Chapter 28, Part 1, of the Manual:

1.1 SPECIAL NOTES

(a) The clearances shown are for tangent track and new construction. Clearances for reconstruction work or for alteration are dependent on existing physical conditions and, where reasonably possible, should be improved to meet the requirements for new construction. Equivalent metric dimensions are shown in parenthesis (millimeters) based on 1′-0″ = (304.8).

(b) On curved track, the lateral clearances each side of track centerline shall be increased 1-½ inches (38.1) per degree of curvature. When the fixed obstruction is on tangent track but the track is curved within 80 feet (24384) of the obstruction, the lateral clearances each side of track centerline shall be increased as follows:

Distance from Obstruction to Curved Track			Increase Per Degree of Curvature	
Feet	(Millimeters)		Inches	(Millimeteres)
20	(6096)	....	1-½	(38.100)
40	(12192)	....	1-⅛	(28.575)
60	(18288)	....	¾	(19.050)
80	(24384)	....	⅜	(9.525)

(c) On superelevated track, the track centerline remains perpendicular to a plane across top of rails. The superelevation of the outer rail shall be in accordance with the recommended practice of the AREA.

(d) In some instances state or Canadian laws and individual railroads require greater clearances than these recommended minimums. Any facility adjacent to or crossing over railroad tracks must not violate applicable state laws, Canadian law, or requirements of railroads using the tracks. As information, a summary of the various state laws is shown in chart form on Page 28-3-25.

COMMITTEE 33—ELECTRICAL ENERGY UTILIZATION

The Committee recommends the following additions to Chapter 33, Part 4, of the Manual:

4.1.6 OVERLOAD OR SHORT TIME AMPACITY CONDITIONS

Electric locomotives and substation transformers normally are given a short time overload rating. This rating may vary from 10 percent for an hour to 200 percent for a few minutes. These overload ratings are possible because locomotives and substation transformers are forced cooled and/or equipped with a large oil bath heat sink. These conditions do not exist with catenary wires.

Catenary wires in calm air will reach their equilibrium temperature from resistance heating in a very short period of time; typically measured in terms of seconds up to perhaps a minute. These time frames are only a small fraction of the time most railroaders associate with the term "overload rating." These short time frames are, however, sufficient to handle short circuits when the circuit breakers are properly set. Short circuit currents of 10 to 30 times normal are typical and will quickly anneal wires if the circuit breakers do not open. It is thus critical to ensure that appropriate maintenance and calibration procedures are followed.

4.1.7 OVERLOAD OR SHORT TIME AMPACITY RATINGS

Due to the relatively short time it takes catenary wires to heat up, it is recommended that *no* overload or short time ratings be used for catenary wire systems except those associated with the clearing of short circuits.

All non-short circuit related currents should be treated as continuous (steady state) conditions covered in paragraphs 4.1.2 through 4.1.5.

4.2 CATENARY SYSTEM DESIGN CRITERIA

4.2.1 GENERAL INTRODUCTION:

The specific detailed design of a railway catenary system will be governed by the interaction of a number of conditions imposed by the climate, vehicle design, desired train operation speeds, electrical loads and conditions, local legal codes, structural material limitations, etc. While the designer must evaluate conditions unique to each site, there are many parameters common to different installations for which general recommendations in this section should be viewed as a starting point from which economic and technical analysis may justify deviations; i.e. ice storm frequency of once a century might justify reducing or eliminating this criteria. Users of these recommendations should also refer to the American National Standard NATIONAL ELECTRICAL SAFETY CODE for further guidance.

4.2.2 CLIMATIC CONDITIONS

4.2.2.1 General:

The following climatic conditions can combine in a number of different combinations to produce a worst case design condition. In the interest of personal and equipment safety most railroad administrations will reduce or suspend operations under extreme storm conditions such as blizzards, hurricanes or massive ice storms. Catenary systems thus have two basic design categories; operating and non-operating or design limiting.

4.2.2.2 Operating Temperature Range:

Although local ambient temperatures have relatively wide variations, solar and resistive heating result in a relatively typical range of maximum operating conductor temperatures of 160-200°F (71-94°C). Constant tension catenary will typically have low temperature mechanical stops placed in the tensioning system to prevent excessive wire movement during abnormally low temperatures. These stops are normally placed at a point where 5% of the daily low temperatures will cause engagement.

4.2.2.3 Icing Conditions:

A coating of ice on the catenary system will significantly increase the total weight being supported and the area subject to wind forces. These increases translate into larger supports and foundations. The additional catenary system costs related to icing conditions should be balanced against the historical frequency of those conditions and the factors of safety used in the designs. The National Electrical Safety Code contains additional details relative to ice loadings. The following criteria are recommended except where frequent train operations would justify reduced radial ice on the contact wire.

A. Radial Ice	0.5 in. (1.3 cm)
B. Wind Speed	40 mph (65 km/hr)
C. Max. Contact wire lateral Deflection at a support	6 in. (15 cm)
D. Temperature	10°F (−12°C)

4.2.2.4 Wind Pressure:

It is recommended that the equation $P = 0.00256CV^2$ be used to calculate wind pressure on wires and structures. The resulting pressure is in pounds per square foot for velocities in miles per hour. The shape factor C is given for typical applications:

A. Wires	1.0
B. Cylindrical Sections	0.8
C. H Sections	1.2
D. Lattice Structures	1.8
E. Flat Surfaces	1.4

4.2.2.5 Wind Speed: The following wind speed criteria are recommended

A. *Operating Wind Speed:* 48-60 mph (77-96) km/hr). This value should be used to compute catenary support and wire deflections for the interface with the vehicle/pantograph system.

B. *Design Wind Speed:* 60-80 mph (96-130) km/hr). This value is used to determine the strength requirements of the catenary system without ice loading.

C. *Exposed Areas:* Railroads placed on very high embankments or very flat exposed areas can be subjected to abnormal wind conditions. Wind speed criteria should be increased by the following factors for exposed areas and high embankments:

1. Operating wind speed	1.25
2. Design wind speed	1.5

D. *Sheltered Areas:* Across track wind speeds in cuts, deep forests or urban areas are less than normal. Wind speeds in sheltered areas can be reduced to 0.8 times that selected for normal operating and design conditions.

4.2.3 CLEARANCE CRITERIA

4.2.3.1 Pantograph size:

As with section 2 of this Chapter, the pantograph is assumed to measure 6ft. 6in. (1.98m) over the tips of the horns with a working width of at least 4ft. 4in. (1.32m) able to effectively contact the wire.

4.2.3.2 Tangent Support Locations:

State and sometimes local codes usually specify minimum distances from track centerline to any lateral obstructions; this is frequently 8ft. 6in. (2.6m). Chapter 28 of this Manual also provides design guidance with respect to bridges and tunnels for electrified railroads. It should be noted that increasing the support offset from the track centerline will increase pole and foundation sizes, cantilever arm size and total system cost. The following offsets from track centerlines to face of poles are recommended:

A. Normal offsets	9ft. 6in. (2.9m)
B. Minimum offsets	8ft. 6in. (2.6m)

4.2.3.3 Curve Adjustments to support locations:

Lateral clearances on curves are normally increased by one inch (2.5cm) for each degree of curvature. Superelevation should be compensated for as shown in part 2.2.6 of Chapter 33. This adjustment is needed only when the minimum offset is being used or when the adjusted clearance would fall below the 8ft. 6in. (2.6m) minimum.

4.2.3.4 Contact Wire Height:

Contact wire height will vary considerably over an entire route being electrified as overhead obstructions are avoided. Minimum contact wire height will be determined by the required load clearances, type of obstruction and insulation techniques employed. Recommended contact wire height in open territory which allows for a future one foot (0.3m) of track raising and surfacing is.

A. 50K system	24 feet (7.3m)
B. 25KV system	23 feet (7.0m)
C. 12.5KV system	22 feet (6.7m)

It should be noted that the National Electrical Safety Code requires a minimum 22 foot (6.7m) clearance at highway grade crossings under the worst conditions. At locations where people may be required to get on top of vehicles (shops or selected yard conditions) the contact wire should be placed near the maximum reach of the pantograph (frequently 25ft. or 7.6m) with grounded isolation capabilities required.

4.2.4 CONTACT WIRE DEFLECTIONS

4.2.4.1 Contact Wire Stagger:

In order to distribute wear across the pantograph wear strips the contact wire is usually pulled from one side of the track centerline to the other alternately from support to support on tangent track. The recommended stagger in both directions from the track centerline is 6 in. (15cm) on tangent track.

4.2.4.2 Contact Wire Stagger on Curves:

The recommended stagger on curves at the support should be placed towards the outside of the curve and should not exceed 6 in. (15cm). Care should be taken on sharp curves to insure that the midspan offset towards the inside of the curve does not exceed 6 in. (15cm).

4.2.4.3 Ambient Temperature Deflections:

The typical cantilever catenary support arms used for constant tension catenary systems rotate through an arc as the wire temperature changes until the low temperature stops are engaged. This along track rotation results in the wire moving perpendicular to the track with the maximum deflection occurring at the tensioning devices. The recommended maximum deflection perpendicular to the track caused by temperature changes is 2.5 in. (6.3cm).

4.2.4.4 Wind Deflection:

High wind conditions will cause both catenary and vehicles to be deflected in the same direction. It is recommended that the maximum pole deflection under operating conditions without ice at the contact wire height be limited to 4 in. (10 cm).

Wind deflection of the contact wire between supports is a function of wire tensions, catenary styles, wind angle, span length, etc. It is recommended that mid-span deflections be limited to 12 in. (30 cm) without ice and 10 in. (25 cm) with ice. These recommendations are based on the theoretical vehicle sway (Chapter 33, Part 2.2.6) against strong winds being effectively reduced by 50 percent.

NOTE: Total contact wire deflection should be adjusted downward if poor track cross-level is present.

4.2.5 CONSTRUCTION TOLERANCES:

The following construction tolerance values (deviations from specified dimensions) are recommended:

A. Main Line Alignment	1 in (2.5cm)
B. Secondary Track Alignment	2 in (5cm)
C. Main Track Elevation	1 in (2.5cm)

D. Secondary Track Elevation	3 in (7.5cm)
E. Pole Location	3 in (7.5cm)
F. Foundation Elevation	1 in (2.5cm)
G. Contact Wire Alignment	2 in (5cm)
H. Contact Wire Elevation	
1. Yard Conditions	6 in (15cm)
2. 30 mph (48 km/hr)	4 in (10cm)
3. 45 mph (73 km/hr)	3 in (7.5cm)
4. 60 mph (96 km/hr)	2 in (5cm)
5. 80 mph (130 km/hr)	1.5 in (3.2cm)
6. 100 mph (160 km/hr)	1.25 in (3.2cm)
7. 125 mph (200 km/hr)	1.0 in (2.5cm)

4.2.6 CONTACT WIRE GRADIENTS:

The rate at which the contact wire changes its elevation relative to the track elevation is very important if pantograph bounce and the resulting arcing are to be avoided. The following recommended maximum gradients will permit the use of multiple locomotives with operating pantographs:

A. Yard Conditions	2.3%
B. 30 mph (48 km/hr)	1.3%
C. 45 mph (73 km/hr)	0.8%
D. 60 mph (96 km/hr)	0.6%
E. 80 mph (130 km/hr)	0.5%
F. 100 mph (160 km/hr)	0.4%
G. 125 mph (200 km/hr)	0.3%

Except for yard conditions, the change of grade from one span to the next should not exceed one half the values shown.

PORTFOLIO RECOMMENDATIONS

COMMITTEE 5—TRACK

The following changes are proposed to improve the Portfolio of Trackwork Plans:

The addition of former Plan 1002-52, Girder Rail Sections, for in-pavement track for light-rail transit systems and other uses.

Plan 1010-82, Permissible Variations in Completed Frogs, will add a note on joint holes.

Plan 1011-82, Permissible Variations in Completed Switches, will add a note on joint holes.

PUBLISHED AS INFORMATION

COMMITTEE 9—HIGHWAY-RAILWAY PROGRAMS

Report of Subcommittee No. 3

Summary Reporting of Significant Publications on Grade Crossing Safety — Report for the Years 1980-1981-1982-Early 1983

F.J. Kull, (Chairman Committee 9), J.R. Summers, (Vice Chairman Committee 9), A.D. Moore, (Secretary Committee 9), Dr. H.L. Michael, (Chairman Subcommittee 3)

Introduction

The Subcommittee continues to report in summary format significant publications or developments in grade crossing safety as directed by its assignment. As the last report of the Subcommittee was for the years 1978 and 1979, this report will cover the years 1980-1981-1982 with some in the early months of 1983. A total of fifteen (15) publications are abstracted and progress on several research projects and developments in other areas influencing grade crossing safety are reported.

The Effectiveness of Flashing Lights and Flashing Lights with Gates in Reducing Accident Frequency at Public Rail-Highway Crossings 1975-1978, J. Morrissey, Input Output Computer Services, Inc., 400 Totten Pond Road, Waltham, MA 02154, Prepared for U.S. Department of Transportation, FRA and FHWA, Washington, D.C. 20590, April 1980, 19 pp.

The Highway Safety Acts of 1973 and 1976, and the Surface Transportation Assistance Act of 1978, provide funds to individual states to improve safety at public rail-highway crossings. This report was undertaken in support of a U.S. DOT effort to develop a resource allocation model designed to select and rank crossings, and recommend warning device improvements in a cost-effective manner.

Input to the model included the effectiveness of active warning devices, flashing lights and flashing lights with gates, in reducing accident potential. The effectiveness is defined as the percentage of accident reduction at crossings which result from the installation of warning devices. Previous effectiveness values were available from a 1974 California Public Utilities Commission study.

This report is based on inventory and accident data available from computerized FRA data bases, and computes new effectiveness values in three categories: (1) flashing lights at formerly passive crossings, (2) flashing lights with gates at formerly passive crossings, and (3) flashing lights with gates at crossings formerly equipped with flashing lights only.

The results indicated that flashing lights and flashing lights with gates significantly reduce the number of accidents at public crossings. In addition, gates proved to be more effective than flashing lights in accident reduction.

One Step Toward Standards of Performance for Warning Signals at Railroad Grade Crossings, Richard A. Mather,, unpublished paper (available from the author at Public Utility Commissioner, Labor and Industries Building, Salem, Oregon 97310), June 1980, 17 pp.

Presently, specific standards for evaluating the effectiveness of warning lights at railroad grade crossings do not exist. Those standards which are on the books are directed toward intermediate factors (such as reflectors and roundels), or special situations (such as in laboratory settings). Neither set of standards applies completely to signal evaluation under field conditions.

A testing unit known as the RAM system is prepared to field test performance of signal units.

This paper represents an initial effort to establish objective testing procedures and standards which can aid in identifying faulty warning light units at railroad grade crossings. Footcandle readings, below those which will constitute "acceptable" output, will flag those units for further attention, maintenance, or repair. This determination will be within the capabilities of an inspector with minimal training in use of the RAM system. This versus the months or years of experience currently required of an inspector if he is to make consistently reliable determinations.

For the first time, quantifiable data regarding signal light output can be collected efficiently under field conditions. In addition to its obvious use for troubleshooting, the RAM system has already proven its worth in collection of certain classes of raw data upon which uniform standards can be based.

Rail-Highway Crossing Warning Device Life Cycle Cost Analysis, Jennifer Heisler and Joseph Morrissey, prepared for U.S. Department of Transportation, FRA and FHWA, Washington, D.C., Input Output Computer Services, Inc, 400 Totten Pond Road, Waltham, MA 02154, September 1980, 96 pp.

The Highway Safety Acts of 1973 and 1976, and the Surface Transportation Assistance Act of 1978 provide funds to individual states to improve safety at public rail-highway crossings. This report was undertaken in support of a U.S. DOT effort to improve the efficient allocation and use of these Federal funds.

The report describes the results of a study designed to collect, analyze, and document life cycle costs of active rail-highway crossing warning devices. Life cycle costs were determined from information on installation costs contained in the final billings of rail-highway crossing improvement projects and from data on maintenance costs provided by various states, railroads, and railway associations.

Life cycle costs were analyzed by cost components for each of the five Federal Railroad Administration regions. Cost components included pre-engineering, labor, material, and equipment rental costs as well as maintenance costs. Cost variability due to several factors such as number of tracks, crossing location, type of train detection system, and combinations of these variables was analyzed.

Life cycle installation costs and maintenance costs were determined for each of the active motorist warning devices, as shown in Table A.

TABLE A. INSTALLATION, MAINTENANCE, AND TOTAL LIFE CYCLE COSTS FOR MOTORIST WARNING DEVICES (IN $K)

Motorist Warning Device	Installation Cost	Maintenance Cost	Total Life Cycle Cost
Flash Lights	26.0	14.1	40.1
Cantilevered flashing lights	29.4	17.4	46.8
Flashing lights with gates	39.2	23.2	62.4

Cantilevered flashing lights with gates	44.6	27.1	71.7
Flashing lights upgraded to flashing lights with gates	33.5	23.2	56.7
Flashing lights upgraded to cantilevered flashing lights with gates	42.7	27.1	69.8

Railroad-Highway Grade Crossings: Not Just an Engineering Problem, Otto F. Sonefeld, TRANSPORTATION RESEARCH NEWS, Number 91, Transportation Research Board, November — December 1980, pp. 7-9.

At the risk of oversimplifying a very complex problem, I would like to suggest three factors that I think are critical to the reduction of grade-crossing problems nationally:

1. There is a need for much greater communication, cooperation, and understanding between the many parties that have responsibility for resolving crossing problems. Those grade-crossing programs that can be labeled successful are universally characterized by constant interaction between all government levels and all railroad levels; the unsuccessful ones are plagued by stubbornness, short-sightedness, and general misunderstanding.

2. There is a need for more and better research on driver behavior. We are still woefully unaware of why rail-highway accidents really occur. One of the keys to understanding this problem involves a much more penetrating analysis of accident data, including both railroad and police reports, and the information available in the national grade-crossing inventory system.

3. My final point is simply to restate the need for driver education programs that are objective, credible, basic, and, most of all, continuous. The railroad industry will continue to do its share in this regard, but the long-range payoff clearly requires the cooperation assistance, and expertise of those organizations, private and public, that for many years have involved themselves in highway safety education.

Safety Effectiveness Evaluation — The Improvement of Nighttime Conspicuity of Railroad Trains, National Transportation Safety Board, Office of Evaluations and Safety Objectives, Washington, D.C., 20594, April 13, 1981, 45 pp.

The National Transportation Safety Board examined nighttime accidents in which highway vehicles strike trains that block grade crossings. There is adequate evidence to suggest that this type of accident is strongly influenced by motorists' inability to perceive the presence of trains in crossings because trains lack conspicuity within their environment. This type of accident results each year in approximately 1,800 collisions with 140 persons killed and 800 injured. The Safety Board reviewed pertinent research undertaken by the Federal Railroad Administration (FRA) on a known countermeasure — reflectorization.

The Safety Board issued recommendations to the FRA to develop and issue an advance notice of proposed rulemaking within 6 months for the improvement of nighttime train car and locomotive visibility at grade crossings to aid in preventing accidents in which motor vehicles run into the sides of trains at night. Additionally, the Board recommended that the FRA cooperate with the Federal Highway Administration, the National Committee on Uniform Traffic Control Devices, and the Association of American Railroads to plan and institute a research program on criteria for the use of reflectorization devices and materials.

Considerations, Costs and Savings to Evaluate when Deciding to Introduce a Highway-Railroad Grade Separation, Andrew J. Ballard, Technical Paper submitted in partial fulfillment of the requirements for the degree of Master of Engineering, Texas A&M University, Civil Engineering Department, April 13, 1981, 33 pp.

After a review of the literature pertaining to warrants for highway-railroad grade separations, a study to identify factors, costs, and savings involved in determining the need for such grade separations was undertaken. Questionnaires were sent to 79 professionals involved in the interaction of highway transportation and rail transportation. Those who responded rated 14 factors on a scale of one to ten in terms of the importance these factors held in the determination of a need for a highway-railroad grade separation. Several respondents supplied the data pool with additional factors to consider, as well. In addition to the survey, suggested dollar figures for costs and savings which can be expected when a grade separation is introduced are reported.

The five factors rated most important in determining the need for grade separation were in the order of importance:

Train and Vehicle Volumes
Train and Vehicle Speeds
Exposure Level
Frequency and Duration of Blockages
Emergency Vehicle Operations

Rail-Highway Crossing Resource Allocation Model, E.H. Farr, U.S. Department of Transportation, Transportation Systems Center, Cambridge, MA 02142, April 1981, 76 pp. (A technical paper covering the above research by E.H. Farr and Betty H. Tustin, titled "Optimizing Resources at Rail-Highway Crossings", was published in ITE JOURNAL, January 1982, pp. 25-28)

This report describes a methodology developed at the Transportation Systems Center for the Federal Railroad Administration and the Federal Highway Administration to aid in determining the most effective allocation of funds to improve safety at rail-highway crossings. One way to improve safety is to install active motorist warning device at all of the 216,000 public rail-highway crossings in the United States, a method has been developed for selectively allocating these funds in an optimal way.

The resource allocation model employs an accident prediction formula which was determined statistically from the extensive data base of the DOT-AAR National Rail-Highway Crossing Inventory and the FRA accident files. The predicted accident rates from this formula, combined with the warning system effectiveness and cost parameters, provide a funding priority ranking of crossing/warning device combination options which are based upon benefit/cost ratios. By selecting from this list, decisions can be made which could increase the accident reduction benefits for any given funding level.

The Development of an Improved Railroad-Highway Grade Crossing Risk Factor, D.E. Scheck, Ohio University, Athen, Ohio 45701 (prepared for the Ohio Department of Transportation), November 25, 1981, 32 pp.

Validation studies were done on five hazard rating models for railroad highway grade crossings. The performance of the Armour Hazard Rating Model was compared to three recently developed models — Federal Highway Administration Florida Department of Transportation and the Canadian Railway Transport Committee — and the well established New Hampshire model. Ohio railroad-highway grade crossing accidents data for 1973 through 1979 were used in this study.

The crossings were ranked in decreasing order of hazard rating for each model. Starting from the top, the cumulative number of crossings and the number of accidents at these intersections were expressed as percentages of all intersections and all accidents respectively. The validation was based on power which is the ratio of cumulative percentage of accidents to the cumulative percentage of crossings.

Consistency of the models was measured by computing the proportion of crossings that were common among the top five percent of each ranking. Commonality within the rankings of the Armour Model and the other four was on the order of fifty percent but when compared to the crossings ranked by decreasing number of reported accidents the average agreement was only 27 percent.

The results indicated that the Armour Model was not markedly better or worse than the other models. The New Hampshire model was one of the top rated models by all measures and was recommended over models that require relatively large and expensive-to-maintain data bases. Approximately one-third of the accident records were unuseable and further work on developing a new or improved hazard rating model was not recommended until the accuracy of accident reporting is greatly improved.

Effects of Rail-Highway Grade Crossings on Highway Users, James L. Powell, Transportation Research Board, Transportation Research Record 841, 1982, pp 21-28.

Basic research into effects of rail-highway grade crossings on highway users was conducted. The overall objective was to investigate improved techniques for estimating nonaccident effects, such as excess delay, user costs, direct energy consumption, and pollutant emissions. Numerical results also were desired. A microsimulation model is developed for analyzing delays due to train blockages at grade crossings not affected by other highway system bottlenecks. An analytic model is then developed to estimate effects of a vehicle slowing at grade crossings with no train present due to rough surface conditions. These models are validated to the extent possible based on field studies. A sensitivity analysis reveals that for most practical applications, train blockages can be analyzed more easily by using simple equations. A sample application of the method is presented in which 385 grade crossings are evaluated from which design options have been selected. The model developed for analyzing effects of a vehicle slowing with no train present is recommended for further applications, although more extensive validation studies are desirable. Numerical results indicate that non-accident costs of grade crossings dominate accident costs in the ratio of about 3.5:1. The effects of a vehicle slowing with no train present dominate effects of train blockages in the ratio of about 2:1. The methods developed are felt to represent a significant improvement over earlier techniques for estimating highway-user effects. The methods can be applied to evaluation of alternatives such as rail relocations, construction of grade-separation structures, and crossing-surface improvements. Areas of further research are also identified.

Improving Safety at Passive Crossings with Restricted Sight Distance, John E. Tidwell and Jack B. Humphreys, Transportation Research Board, Transportation Research Record 841, 1982, pp 29-36.

Investigations were conducted regarding driver knowledge of grade-crossing information, the relation of driver behavior to driver knowledge, and techniques for advisory speed signing. A driver behavior-oriented method for evaluating passive crossings was developed. It was found that most drivers believe that all crossings regularly used by trains have active protection. Driver performance at the sites observed was not related to driver knowledge of grade-crossing facts. It was noted that drivers who looked for trains did not look a proper distance from the crossing or at an appropriate speed to be considered safe. A procedure was developed to assign safe speeds and to locate signing on the approach to the passive grade crossing. Suggestions are made for areas of future investigation.

The Highway Crossing Program: Uncertainty and Change, C.L. Amos, RAILWAY TRACK AND STRUCTURES, May 1982, pp 18-20.

For the third consecutive year, a record low has been recorded for fatalities involving motor vehicle accidents at public grade crossings. Preliminary figures for 1981 from the Federal Railroad Administration indicate that 623 persons died in these accidents, down from 708 in 1980 (see chart) — a decrease of 12%. Total accidents also declined dramatically from 9,422 to 8,528 — a reduction of almost 13%.

This remarkable improvement has been largely the result of the accomplishments made by the national grade crossing safety improvement program funded under Section 203 of the 1973 Highway Act. Coupled with "Operation Lifesaver", which supplements and compounds the benefits of engineering improvements, this is the most successful highway safety program ever created.

Because of the continuing efforts of the federal government, the states, the railroads, the supply industry, the National Safety Council, and many others, grade crossing safety continues to show significant progress. And, even though many challenges await, confidence remains that we will witness further dramatic improvements.

Highway Grade Crossing Surfaces: A Look Back; The View Ahead, William J. Hedley, RAILWAY TRACK AND STRUCTURES, May 1982, pp 21-22.

From where we stand now, a look back over the history of highway grade crossing developments provides us with some guidance for the view ahead. Our past experience has shown us some successes and some failures. On balance, there was improvement. Many of today's grade crossings have smoother and safer surfaces.

The view ahead is one of expectation of further improvements to grade crossing surfaces following the trends and experiences of the recent past. This, of course, assumes no curtailment of funds allocated to this phase of highway construction activity. A "Quick Guide to Modern Grade Crossing Surfaces", 1982 edition, is included.

A Survey of Train-Activated Advance Warning Devices in Oregon, Richard A. Mather, unpublished paper presented to the AAR Communication and Signal Division Meeting. (Available from author at Public Utility Commissioner, Labor and Industries Building, Salem, Oregon 97310), October 1982, 17 pp.

In 1968, Oregon began to install Train-Activated Advance Warning Devices (TAAWDs) near selected railroad crossings. All existing TAAWDs are installed in conjunction with Train-Activated Warning Devices at the crossing.

The TAAWD is generally installed and maintained by the road authority. The older TAAWDs have only one 110-volt AC 8-inch beacon mounted in the 66″ × 36″ sign. The new standard now has two 110-volt AC 8-inch beacons above the sign with two 150 W-Par 38 floodlights on a mast arm. A two-wire closed circuit loop from the railroad housing activates the TAAWD.

Present per-unit charges associated with post-mounted signs total approximately $1,500. For mast-mounted (cantilevered) signs, the per unit costs are approximately $5,000 installed.

The purpose of this paper is only an attempt to document the physical characteristics of the TAAWDs currently in operation throughout the state. It is not the intent to critically analyze or to question the rationale with regard to why certain TAAWDs are installed as they are. Nor is judgment passed on the effectiveness of various configurations.

Railroad/Highway Grade Crossing Accidents Involving Trucks Transporting Bulk Hazardous Materials, Lawrence E. Jackson, ITE JOURNAL, October 1982, pp 35-37.

In the last 15 years, the NTSB has investigated 17 accidents of this type, but 11 of these investigations have been conducted in the last two years.

In the more recently investigated accidents, the NTSB observed two common factors. The first factor was that these accidents tend to occur near a bulk material terminal.

The second factor was that drivers involved in these accidents appeared to demonstrate an irresponsible or careless attitude at the crossings. The FRA data indicate that 30% of all the truck accidents occurred because the drivers did not obey flashing lights or gates. In another 41% of the truck accidents, the drivers failed to stop at a passive crossing and did not perceive the approaching train.

Proceedings, National Conference Railroad Highway Safety, Kansas City, Missouri, August 31 — September 2, 1982, Edited by Hoy A. Richards (available from National Technical Information Service) December 1, 1982, 174 pp.

This edited Proceedings includes summaries fo the activities of eight (8) sessions held at the 1982 Conference. The subjects of these sessions were:

SESSION A: Current Legislation
SESSION B: Current Research
SESSION C: Creditability, Reliability and Maintenance of Warning Devices
SESSION D: Public Education: Operation Lifesaver
SESSION E: Laws, Regulations and Standards
SESSION F: Human Factors in Rail/Highway Crossing Evaluation
SESSION G: New Ideas in Program Implementation
SESSION H: Workshops

Copy of several papers presented at each session is included.

OTHER GRADE CROSSING ACTIVITIES

Operation Lifesaver

Operation Lifesaver developed strongly in a number of states during the early 1980s. Over 30 states had active programs in 1982 with developments likely in other states during 1983. This fine educational effort has proved to be another valuable effort in development of increased awareness and respect of citizens of all ages to motorists responsibilities at rail-highway grade crossings. The national and state direction of the National Safety Council and promotion by railroad authorities in the program has provided continuing leadership for this effort.

A published report on the "1981 Operation Lifesaver National Symposium" held August 27-29, 1981, is available from the National Safety Council. This 40 page publication reviews past and current Operation Lifesaver programs, defines today's issues and problems and develops methods and techniques which will increase driver awareness of hazards at rail-highway crossings. Quality programs and effective efforts are reviewed and issue workshop reports are included.

National Committee on Uniform Traffic Control Devices

The Rail-Highway Grade Crossing Technical Committee of this national group reviewed several requests for changes in the National Manual on Uniform Traffic Control Devices (MUTCD) over the past three years and made a recommendation to the Federal Highway Administration on each of them. Notable in changes approved which were recommended was the addition to the National MUTCD of new standard advance warning signs for railroad crossings for motorists on roads parallel to railroads on an approach to the crossing. Criteria as to where the new signs shall be used and placement of them are also included.

The Technical Committee meets twice each year (in January and July) to consider requests for changes from the public as well as changes in the MUTCD proposed by its members and study groups. The National Committee and its Technical Committee have developed into a fine cooperative group sponsored by sixteen (16) national organizations interested in highway traffic control devices. It has an excellent working relationship with the Federal Highway Administration and the future outlook for continued improvement of the MUTCD by FHWA with the technical counsel of the National Committee appears excellent.

Published FRA and FHWA Reports on Rail-Highway Crossing Safety 1969-1982

A very useful listing of "Rail-Highway Crossing Reports 1969-1982" made by FRA and FHWA was compiled by RRS-12, Office of Safety, Federal Railroad Admininstration, July 1983, and is an attachment to this Subcommittee Report. Information on report availability and cost of some of the reports reviewed in this Subcommittee Report and in reports of earlier years is given.

CONCLUDING COMMENTS

A large amount of research is currently in progress or has been completed within the past year with final reports yet to be published. As a consequence, it is recommended that the assignment of the subcommittee continue without change. The summary of important publications provided by the subcommittee annual report should continue to be valuable to railroad-highway grade crossing safety among AREA members.

ATTACHMENT A

RAIL-HIGHWAY CROSSING REPORTS, 1969-1982

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL RAILROAD ADMINISTRATION

I. Ordering Information:

Reports having an NTIS Order Number may be ordered from National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161. Enclose check or money order. Prices are effective as of January 1, 1982, and are subject to change thereafter. For other reports, a more comprehensive listing may be found in Bibliographies 57 and 58, published by and available through the Transportation Research Board, Washington, D.C. 20418

II. Published FRA Reports in Chronological Order:

		NTIS Order No.	NTIS Price	Price Code
1.	*A Program Definition Study for Rail-Highway Grade Crossing Improvement*. Prepared by Alan M. Voorhees and Associates, Inc., for FRA, Report FRA-RP-70-2, October 1969.	PB 190401	$16.00	A08
2.	*The Visibility and Audibility of Trains Approaching Rail-Highway Grade Crossings*. Prepared by Systems Consultants, Inc., for FRA, Report FRA-RP-71-1, May 1971.	PB 202668	16.00	A08
3.	*Technological Innovation in Grade Crossing Protective Systems*. Prepared by TSC for FRA, Report DOT-TSC-71-3, June 1971	PB 201624	11.50	A05
4.	*Report to Congress. Railroad-Highway Safety*. Part I: A Comprehensive Statement of the Problem. Part II: Recommendations for Resolving the Problem. Prepared jointly by the Staffs of FRA and FHWA. November 1971 and August 1972.	PB 206792 PB 213115	14.50 13.00	A07 A06
5.	*Human Factors Countermeasures to Improve Highway-Railway Intersection Safety*. Prepared by Bio Technology, Inc. for FRA and NHTSA. Report No. DOT-HS-800888, July 1973, 235 pp. PB 223416. Price: $19.50 A11.			
6.	*Enhancement of Train Visibility*. Prepared by TSC for FRA, Report FRA-ORD&D-74-15, September 1973.	PB 223899	11.50	A05
7.	*Grade Crossing Protection in High-Speed, High-Density, Passenger-Service Rail Corridors*. Prepared by TSC for FRA, Report FRA-ORD&D-74-14, September 1973	PB 223738	8.50	A03
8.	*Proceedings 1974 National Conference on Railroad-Highway Crossing Safety*. Sponsored by U.S. Department of Transportation. U.S. Air Force Academy, August 19-22, 1974.	PB 245231	13.00	A06
9.	*State Grade Crossing Programs: A Case Study*. Prepared for FRA/TSC by CONSAD Research Corp., Report FRA-ORD&D-75-8, September 1974.	PB 244175	10.00	A04
10.	*Feasibility Study of In-Vehicle Warning Systems*. Prepared by Tracor-Jitco for NHTSA/FRA, Report DOT-HS-256-3-752, March 1975.	PB 242462	14.50	A07
11.	*Field Evaluation of Locomotive Conspicuity Lights*. Prepared by TSC for FRA, Report FRA-ORD&D-75-54, May 1975.	PB 244532	10.00	A04

		NTIS Order No.	NTIS Price	Price Code
12.	*Guidelines for Enhancement of Visual Conspicuity of Trains at Grade Crossings.* Prepared by TSC for FRA, Report FRA-ORD&D-75-71, May 1975.	PB 244551	10.00	A04
13.	*A Communication-Link Approach to Actuation of Grade Crossing Motorist-Warning Systems.* Prepared by TSC for FRA, Report FRA-ORD&D-75-80, July 1975.	PB 244584	13.00	A06
14.	*A Methodology for Determination of Grade Crossing Resource Allocation Guidelines.* Prepared by TSC for FRA, Report FRA-ORD&D-76-04, August 1975.	PB 259005	10.00	A04
15.	*Locomotive to Automobile Baseline Crash Tests.* Prepared by Ultra Systems for FRA, Report FRA-ORD&D-76-03, August 1975.	PB 250564	14.50	A07
16.	*Lightning and Its Effects on Railroad Signal Circuits.* Prepared by University of Lowell, Lowell, MA., for FRA/TSC, Report FRA-ORD&D-76-129, December 1975.	PB 250621	13.00	A06
17.	*Standby Power for Railroad-Highway Grade Crossing Warning Systems.* Prepared by University of Lowell, Lowell, MA., for FRA/TSC, Report FRA-ORD&D-76-286, September 1976.	PB 263592	8.50	A03
18.	*Improvement of the Effectiveness of Motorist Warnings at Railroad-Highway Grade Crossings.* Prepared by TSC for FRA, Report FRA/ORD-77/07, February 1977.	PB 266784	11.50	A05
19.	*Potential Means at Cost Reduction in Grade Crossing Automatic Gate Systems.* Prepared by MB Assoc., et. al for TSC/FRA, Reports FRA/ORD 77-06.I and 77-06.II, February 1977.	PB 265724 PB 265725	11.50 10.00	A05 A04
20.	*Summary Statistics of the National Railroad-Highway Crossing Inventory for Public At-Grade Crossings.* Prepared by TSC for FRA, Report FRA-OPPD-77-8, June 1977.	PB 271334	16.00	A08
21.	*Proceedings 1977 National Conference on Railroad-Highway Crossing Safety.* Sponsored by U.S. Department of Transportation. University of Utah, August 23-25, 1977	PB 293071	14.50	A07
22.	*Innovative Concepts and Technology for Railroad-Highway Grade Crossing Motorist-Warning Systems.* Prepared by Cincinnati Electronics et. al for TSC/FRA, Reports FRA/ORD-77/37.I and 77/37.II, September 1977.	PB 273354 PB 273355	19.00 11.50	A10 A05
23.	*Potential Means of Cost Reduction in*	PB 277946	17.50	A09

		NTIS Order No.	NTIS Price	Price Code
	Grade Crossing Motorist-Warning Control Equipment. Prepared by Storch Engrs. et. al for TSC/FRA Reports FRA/ORD-77/45.I and 77/45.II, December 1977.	PB 277947	8.50	A03
24.	*Analysis of NPRM Strobe Lights on Locomotives*. Prepared by IOCS, Inc., for FRA, Report FRA-OPPD-79-4, May 1978	PB 293483	10.00	A04
25.	*A Study of State Programs for Rail-Highway Grade Crossing Improvements*. Prepared by TSC for FRA, Report FRA-OPPD-78-7, June 1978.	PB 279774	16.00	A08
26.	*Summary Statistics of the National Railroad Highway Crossing Inventory for Public At-Grade Crossings, Second Edition*. Prepared by TSC for FRA, Report FRA-OPPD-78-20, September 1978.	PB 293070	16.00	A08
27.	*Legal Effects of Use of Innovative Equipment At Railroad-Highway Grade Crossings on Railroad's Accident Liability*. Prepared by TSC for FRA, Report FRA-RRS-80-01, October 1979.	PB 80-137888	10.00	A04
28.	*Rail-Highway Crossing Accident/Incident and-Inventory Bulletin (No. 1 Calendar Year 1978)*. Prepared by Office of Safety, FRA, October 1979.	Available directly from FRA.		
29.	*Rail-Highway Crossing Hazard Prediction Research Results*. Prepared by TSC for FRA, Report FRA-RRS-80-02, March 1980.	PB 80-170749	20.50	A11
30.	*Identification and Evaluation of Off-Track Train Detection Systems for Grade Crossing Applications*. Prepared by GARD, Inc. for FRA, Report FRA/ORD-80-32, April 1980.	PB 80-186430	13.00	A06
31.	*Operational Testing of Locomotive-Mounted Strobe Lights*. Prepared by TSC for FRA, Report. DOT-TSC-FRA/ORD-80-48, June 1980.	PB 80-224348	8.50	A03
32.	*The Effectiveness of Flashing Lights and Flashing Lights with Gates in Reducing Accident Frequency at Public Rail-Highway Crossings, 1975-1978*. Prepared by Input Output Computer Services, Inc. for TSC/FRA/FHWA, Report FRA-RRS-80-005, April 1980.	PB 81-133886	8.50	A03
33.	*Rail-Highway Crossing Warning Device Life Cycle Cost Analysis*. Prepared by Input Output Computer Services, Inc. for TSC/FRA/FWHA, Report FRA-RRS-80-003, September 1980.	PB 81-133894	11.50	A05
34.	*Rail-Highway Crossing Accident/Incident and Inventory Bulletin (No. 2 Calendar Year 1979)*. Prepared by Office of Safety, FRA, September 1980.	Available directly from FRA.		

	NTIS Order No.	NTIS Price	Price Code
35. *Grade Crossing Accident Injury Minimization Study.* Prepared by HH Aerospace Design Company, Inc., for FRA, Report No. FRA/ORD-80-87, December 1980.	PB 81-155236	17.50	A09
36. *Proceedings, 1980 National Rail-Highway Crossing Safety Conference.* Sponsored by U.S. Department of Transportation, University of Tennessee, June 17-19, 1980.	PB 81-196982	13.00	A06
37. *Investigation of Anomalous Rail-Highway Crossings.* Prepared by Office of Safety, FRA, January 1981. Paper presentation at 60th Annual Meeting of TRB. No report number.	Available directly from FRA.		
38. *Rail-Highway Crossing Resource Allocation Model.* Prepared by TSC for FRA/FHWA, Report No. FRA-RRS-81-001, April 1981.	PB 81-212540	11.50	A05
39. *Constant Warning Time Concept Development for Motorist Warning at Grade Crossings.* Prepared by Systems Technology Laboratory, Inc., for FRA, Report No. FRA/ORD-81/07, May 1981.	PB 81-205684	17.50	A09
40. *Rail-Highway Crossing Accident/Incident and Inventory Bulletin (No. 3 Calendar year 1980).* Prepared by the Office of Safety, FRA, June 1981. No report number.	Available directly from FRA.		
41. *Operation Lifesaver National Symposium,* (Aug. 27-29, 1981, Chicago, IL). Prepared by FRA/Office of Safety and National Safety Council, August 1981. No report number.	PB 83-117234	8.50	A03
42. *Rail-Highway Crossing Accident/Incident and Inventory Bulletin (No. 4 Calendar Year 1981).* Prepared by Office of Safety, FRA, June 1982. No report number.	Available directly from FRA.		
43. *Summary of the Department of Transportation Rail-Highway Crossing Accident Prediction Formulas and Resource Allocation Model.* Prepared by TSC for FRA, Report No. DOT-TSC-FRA-82-1. September 1982.	PB 83-119057	8.50	A03
44. *Freight Car Reflectorization.* Prepared by TSC for FRA, Report No. FRA-RRS-83-1, December 1982	Number not yet available		

III. Published FHWA Reports

Federal Highway Administration
Office of Research — HRS-33
400-7th Street, SW
Washington, DC 20590

	NTIS Order No.	NTIS Price	Price Code
1. Motorists' Requirements for Active Grade Crossing Warning Devices/FHWA-RD-77-167. (A limited number of copies are available now from FHWA. Also available from NTIS.)	PB-296 183/AS	22.00	A12
2. Railroad Grade Crossing Passive Signing Study — Final Report/FHWA-RD-78-34. (Available now from NTIS.)	PB-286 528/AS	10.00	A04
3. Safety Features of Stop Signs at Rail-Highway Grade Crossing Vol. 1 — Executive Summary/ FHWA-RD-78-40. Vol. 2 — Technical Report/ FHWA-RD-78-41. (A limited number of copies are available from FHWA. Also available from NTIS.)	PB-295-422/AS PB-295-423/AS	7.00 16.00	A02 A08
4. Experimental Design for Active Grade Crossing Warning Devices/FHWA-RD-77-167. (A limited number of copies are available now from FHWA. Also available from NTIS.)	PB-80-145 089	10.00	A04
5. Railroad-Highway Grade Crossing Handbook/ FHWA-TS-78-214. (Available now from FHWA, 400-7th Street, SW (TAD 443.1) Washington, DC 20590.			
6. Rail-Highway Crossing Accident Causation Study, Vol. I, Executive Summary/ FHWA/RD-81-082. (NTIS number not available.)	PB-83158725	8.50	A03
7. Rail-Highway Crossing Accident Causation Study, Vol. II, Technical Report/FHWA/RD-81/083), (NTIS number not available.)	PB-83158733	14.50	A07
8. *Activated Advance Warning for Railroad Grade Crossings*/FHWA/RD-80/003. (NTIS number not available.)	PB-83161869	17.50	A09
9. *Proceedings 1982 Railroad-Highway Safety National Conference*, Kansas City, MO, August 31-September 2, 1982.	PB-83163501	19.00	A10
10. *Rail-Highway Crossing Resource Allocation Procedure, User's Guide*, prepared by TSC for FHWA and FRA, Report No., FHWA-IP-82-7, December 1982.	Number not yet available		

This list prepared by RRS-12, Office of Safety, Federal Railroad Administration, July 1983.

COMMITTEE 13—ENVIRONMENTAL ENGINEERING

Report of Subcommittee No. 3 (1982-1983)

The Disposal of Discarded Railroad Wood Cross Ties

W.M. Cummings (Subcommittee Chairman), R. Alderfer, E. Hockensmith, G.W. Lawrence, G. Mason, W.C. Studabaker, M.L. Williams, H. Wyche

Railroads are currently incurring costs to dispose of approximately 20 million cross ties each year, but are receiving revenue for only about one-third that number. This indicated that further study of tie disposal methods was needed to improve the revenue/cost ratio.

A study was performed for the railroad industry to develop costs and revenues associated with various tie disposal methods and to investigate the environmental constraints on these methods. The study also develops an investment alternative model to assist railroads in selecting the most cost effective disposal method for each application.

The study reviews tie disposal methods including sale of whole ties for use in landscaping, on-site controlled and open burning, landfilling, sale for use as boiler fuel, shredding on the right-of-way, and re-use for reconstituted wood products. Regulatory constraints on each alternative are discussed and representative capital and operating costs associated with each method are provided.

The investment alternative model provides a vehicle to compare the disposal methods on an annualized unit cost basis for a particular site. The model can be used by a railroad to develop the most cost effective combination of tie removal and disposal techniques for their system. In addition, the model can be used to evaluate future disposal alternatives as they develop. The model will be produced in a micro-computer suitable format.

Based on the findings of this study, individual railroads should attempt to develop larger markets for: 1) whole ties suitable for landscaping and other related purposes, and 2) tie butts for use as fuel. The fuel market is highly underdeveloped and may present the most viable alternative for solution of the tie butt disposal problem. A re-examination of the widespread practice of removing ties from track in thirds may also be warranted.

This publication entitled "The Disposal of Discarded Railroad Wood Cross Ties — A Method of Analyzing Alternatives" is available from the Association of American Railroads, Technical Center, 3140 South Federal Street, Chicago, Illinois 60616.

Report of Subcommittee No. 8

Environmental Regulations and the Railroads

P.C. Conlon (Subcommittee Chairman), R.S. Bryan, W.R. Brunzell, J.C. Dretz, R.E. Fronczak, R.G. Michael, R.J. Spence, T. Pendergast, W. Peters, D.R. York

Introduction

As corporate citizens, railroads must comply with environmental laws and regulations. These are intended to protect the quality of the environment by preventing pollution and conserving resources. Since railroads are corporations, and corporations are people, it is the responsibility of railroad people to control the possible sources of pollution on the railroad. More specifically, the responsibility of controlling pollution lies with those persons directly in charge of a facility or piece of equipment. The purpose of this paper is to briefly describe the federal laws that apply to pollution control. The obligations of environmental protection can be put in perspective with the job of operating and maintaining a railroad only by understanding the scope of the laws and the liabilities associated with them.

Before discussing the federal laws, it should be pointed out that each state and often cities and counties have their own environmental regulations. The requirements of these regulations are often more stringent than the federal laws, particularly in areas of severe pollution problems.

Water Pollution

The nation's surface water is a highly visible and intensely used resource. Less visible but equally important are the nation's groundwater resources. The Federal Water Pollution Control Act, as amended in 1972, commonly called the Clean Water Act, is one of the earliest pieces of modern federal legislation which addresses water pollution control. Its objective was to clean all waters until they were satisfactory for swimming and for fish and wildlife, and to end the discharge of harmful waste into our country's surface waters. Groundwater pollution did not receive much attention at the federal level until the mid 1970s when the Resource Conservation and Recovery Act of 1976 (RCRA) was passed to prevent waste materials from polluting groundwater. Other legislation passed during this period was the Safe Drinking Water Act (1974) which set standards for protecting the quality of drinking water supplies. More recently the Comprehensive Environmental Response and Liability Act (1980) (CERCLA), more commonly referred to as Superfund, was passed to clean up areas of waste contamination, including groundwater.

Every facility, public or private, that discharges wastes into the waters of the United States must have a permit to do so. These permits, known as National Pollutant Discharge Elimination System (NPDES) permits, are required by the Clean Water Act. An NPDES permit generally places limitations on the concentrations of specific pollutants which can be discharged in a wastestream; establishes schedules for upgrading controls to meet such limits; and requires monitoring and periodic reports on compliance with the limitations. Most of these compliance reports are based on samples taken from the wastewater being discharged and are analyzed by persons working for or hired by the discharging facility.

Many railroad facilities release their liquid wastes to a publicly owned wastewater treatment plant rather than directly to a waterway. EPA requires that industrial wastes that are not compatible with municipal treatment plants be pretreated by the industry before they are discharged to the municipal facility. The intent of this requirement is to protect the treatment plant from pollutants which could interfere with the proper operation of the plant or would pass through the plant without proper treatment and removal. Typical wastes that are treated in railroad wastewater treatment plants include spilled diesel fuel and lubricating oil, soaps from wash rocks, sand, dissolved heavy metals such as chrome and lead, and caustic cleaning agents. Most municipal wastewater plants are not designed to remove these pollutants from a wastestream; therefore, railroads must install their own treatment or pretreatment facilities.

The Clean Water Act also prohibits the spilling or leaking of oil and hazardous substances into surface waters or on the adjoining shoreline. EPA requires major oil storage facilities to have written instructions called "Spill Prevention Control and Countermeasure Plans", usually referred to as SPCC plans, readily accessible in case of an emergency spill. The SPCC plan is required to establish procedures, methods and equipment to prevent discharge of oil and hazardous substances from onshore and offshore facilities. In the event of a spill, implementation of the plan insures safe, immediate containment of the spilled material to a limited area, thus preventing widespread adverse effects to the environment. Guidelines for removal and disposal of the spilled substances are also contained in an SPCC plan.

The National Contingency Plan, prepared by EPA, provides guidance in case of spills of oil or hazardous substances into the environment. Each hazardous substance is assigned a "Reportable Quantity," or RQ. This is a quantity which, if spilled in that amount or greater, must be reported to the National Response Center. Any oil spill resulting in a sheen on the water's surface must also be reported. In addition, a system of penalties and liabilities is associated with the spillage of a reportable quantity of such a material, and also with failure to report the spill. Each state has additional reporting requirements. Diesel fuel spills onto land can also pose some difficult pollution abatement problems. Such spills can originate from storage tanks, underground piping, fueling facilities, or derailments. Long-term leaks can result in large quantities of oil in the ground; it is extremely difficult to recover more than about half of the total amount lost because fuel oil absorbs to soil particles. Groundwater contamination may occur when diesel oil comes in contact with a drinking water aquifer. Cleanup of groundwater can be very costly. Frequent inspection of fueling facilities can minimize the likelihood of such accidents as well as an installation of track pans to capture and collect spillage.

Air Pollution

Improving the quality of the nation's air is the goal of the federal Clean Air Act. The Act gives states the primary responsibility for the prevention and control of air pollution. The federal EPA, however, has broad standard-setting, review, and enforcement responsibilities.

The Clean Air Act establishes national primary and secondary air quality standards for seven pollutants: particulates, sulfur dioxide, nitrogen dioxide, ozone, carbon monoxide, lead, and hydrocarbons. Each standard has a primary component designed to protect human health and secondary component designed to protect vegetation as well as personal comfort and well-being.

Each state is required to obtain compliance with these standards through adoption of state implementation plans or SIPs. These plans are basically packages of control laws and regulations governing air pollution and may consist of general statewide prohibitions such as high-sulfur fuel or particular limits on specific sources. The plans also must include regulations aimed at the prevention of significant deterioration of the air quality in those areas complying with air quality standards. In areas not complying with the standards, increases in emissions from an individual plant are permissible only if they are offset by reductions of the same pollutant elsewhere in the plant.

In addition to establishing air quality standards, the Clean Air Act establishes national standards prescribing maximum emission limits for the emission of particular pollutants from particular types of industrial processes and motor vehicles. There are two categories of emission standards — one for stationary sources such as coal-fired boilers — and one for mobile sources such as automobiles.

Railroads are affected by the Clean Air Act in several ways. Many states require control of volatile organic compounds (VOC) found in paints and solvent degreasers. In addition, the emission of hydrocarbons from petroleum products is also regulated, although diesel fuel is generally not affected. Shops performing spray painting or degreasing operations may be required to cut emissions of VOC by controlling evaporation from parts cleaning operations and by using low solvent paint. Petroleum storage facilities are required to meet hydrocarbon emission limits.

Open burning of crossties, trash, and scrapped freight cars is generally prohibited around the country. Reduction of particulate emissions is the prime reason for such bans.

Many states control sulfur emissions by placing restrictions on sulfur content in diesel fuel.

In some areas of the country, dust emissions are regulated. Primary sources include grain, iron ore, and coal loading and unloading, and unpaved roads.

Solid and Hazardous Waste Management

The Resource Conservation and Recovery Act of 1976 (RCRA) is a complex piece of legislation aimed at eliminating open dumping of trash and garbage and, more importantly, the generation, treatment, storage, transportation and disposal of hazardous wastes. As the title of the Act implies, recycling of waste materials is a primary objective of the law. In addition to phasing out open trash dumps, the RCRA creates a "cradle to grave" waste management system intended to ensure that the generated hazardous waste is safely treated, stored, transported and disposed. EPA has identified hazardous wastes both by manufacturing process, commercial name (for products that are discarded), and by the chemical characteristics of ignitability, corrosivity, reactivity and toxicity.

A manifest system designed to track the movement of hazardous waste has been developed by EPA. It requires hazardous waste generators and transporters to employ appropriate management practices as well as procedures to ensure the effective utilization of the manifest system. Generators must make sure that a waste is properly transported to a licensed treatment, storage, or disposal facility. The manifest, prepared by the generator, must accompany the waste to its final destination. The operator of that facility must return a copy of the manifest to the generator after the waste has been received.

Owners and operators of treatment, storage and disposal facilities must comply with EPA standards to protect human health and the environment. Such requirements include operator training, insurance, spill contingency plans, groundwater protection, monitoring, and fencing. These standards are generally implemented through permits that are issued by EPA or states authorized to administer the program. Owners and operators of treatment, storage and disposal facilities are responsible for compliance with the permit requirements.

Hazardous waste generated by a railroad at one facility may be exempted from the RCRA requirements if the quantity is less than 1,000 kg per month. This limit may eventually be reduced to zero. EPA must first gain the practical experience necessary to manage such a huge program. Wastes may not be stored longer than 90 days on the property, however, without obtaining a permit. The 90 day period begins when the first waste is accumulated.

Used oil is not specifically listed as a hazardous waste. However, if the oil meets any of the four hazard characteristics and is not being recycled or burned as a fuel, it is a hazardous waste and subject to the storage and transportation regulations.

All hazardous waste transporters are required to obtain a transporter identification number from EPA. Transporters must comply with the manifest system, establish record keeping procedures, and provide for the cleanup and notification of hazardous waste spills during transit. The regulations permit the use of shipping papers to substitute for the manifest required by RCRA if the shipment is by rail only.

Toxic Substances

Encouragement of better and earlier testing of chemicals for possible adverse effects on human health and the environment was the driving force behind passage of the Toxic Substances Control Act (TOSCA). The law requires any person who manufactures a new chemical substance or who processes an existing substance for a significantly different new use to file an advance notification accompanied with test data to EPA. The key purpose of this prenotification is to enable EPA to evaluate new chemicals before they reach the marketplace. TOSCA also authorizes EPA to review health and environmental risks of the distribution in commerce, use, or disposal of toxic substances. EPA may prohibit or place limitations on any substance it finds to have unreasonable risks associated with its use. EPA can require that any substance be marked with warnings and instructions for use, distribution, or disposal. In addition, EPA requires that records be kept and tests be made to assure compliance with the law.

The most important aspect of TOSCA for railroads are regulations on polychlorinated biphenyls (PCBs). PCB's are used as insulating fluids in many electric transformers. The concentration of PCBs in transformers used in railroad locomotives or railroad self-propelled cars is regulated under TOSCA. These regulations affect only a small portion of the railroad industry which utilize PCB transformers in rolling stock. The regulations require the eventual replacement of PCBs with another form of coolant.

Stationary PCB transformers, e.g. pole transformers used in communication and signal work, with PCB concentrations greater than 50 ppm must be inspected at least once every three months, leaks recorded, and serviced where appropriate. These requirements are currently under review by the EPA which may modify them shortly.

Hazardous Materials

Another important federal statute affecting the railroads is the Hazardous Materials Transportation Act. This law places numerous responsibilities on shippers and transporters of hundreds of hazardous materials and hazardous wastes. Strictly speaking the statute is more in the nature of a safety law since its primary purpose is the safe transport of hazardous materials and wastes. Included in these rules are requirements for packaging of materials, placarding of cars, and safety devices on tank cars such as shelf couplers and head shields.

Environmental concerns arise under the Hazardous Materials Act whenever there is a hazardous materials incident resulting in a discharge from a railcar. Whenever a discharge in a reportable quantity occurs, the railroad is generally required to provide immediate notification by telephone of the incident to the National Response Center and state agencies. Subsequent written reports may also be required.

Most importantly, the railroad is obligated to take whatever cleanup actions are necessary to ensure that the discharge does not present a hazard to human health or the environment.

If an environmental or health problem appears evident, the federal government may consider using its broad powers granted under the Comprehensive Environmental Response, Compensation and Liability Act, or the Superfund, referred to earlier. This legislation provides the federal government with virtually unlimited response authority to address spills of hazardous substances into the environment. It imposes taxes on crude oil and certain chemicals to create a multibillion dollar superfund to pay for the cost of cleanup of these incidents.

Superfund imposes an obligation to notify the National Response Center of any release of hundreds of substances designated as hazardous. This includes hazardous wastes under RCRA, hazardous substances under the Clean Water Act and hazardous air pollutants under the Clean Air Act. There is also a requirement to notify the EPA of inactive hazardous waste storage, treatment, or disposal sites.

EPA must look first to the owner or operator of a facility from which a release occurs for the removal and remedial actions associated with the release. The owner or operator is also liable for any damages to natural resources owned by the federal government or any state governments. If the owner or operator is incapable of assuming financial responsibility for clean-up actions, any previous owner or operator of the facility, the generator of the hazardous waste or transporter who selected the disposal site may be liable.

The EPA will use the superfund primarily where these potentially responsible owner or operator parties are unknown or are without the financial capability to cover removal and remedial action costs.

Noise

The Noise Control Act of 1972 required EPA to set noise emission standards for railroad equipment and operations. The law established federal preemption over state and local noise ordinances. EPA has set noise measurement standards and emission limits for several railroad operations and pieces of equipment, both stationary and moving. Moving railcars are regulated by limits on their sound levels at speeds of up to 45 mph, and at speeds of 45 mph and greater. Idling moving locomotives are subject to noise standards which differ, depending on whether the unit was built before or after 1979. Also, car retarder operations, car coupling operations, locomotive load cell testing and switcher locomotives are subject to EPA standards. Primary enforcement responsibility for these standards rests with the Department of Transportation, Federal Railroad Administration.

Pesticides

The Federal Fungicide, Rodenticide and Insecticide Act (FIFRA) was enacted to control the thousands of pesticide products used in the United States. This law is also administered by the EPA and requires all U.S. pesticides to be registered or approved by the Federal Government. Pesticides are classified for general or restricted use — those that are restricted to be applied only by or under the supervision of certified applicators due to the hazards of the product to humans or the environment. The states regulate the licensing of applications. EPA has established packaging and labelling standards and regulates container disposal. The Act prohibits the misuse of pesticides and EPA enforces these rules with civil and criminal penalties.

Examples of pesticides in use on railroads today include fungicides such as creosote and pentachlorophenol and herbicides such as 2, 4-D, Amizol, Tordon, MSMA, Sodium Chlorate, and Spike.

Basic pesticide handling and container disposal practices recommended by EPA include:

- Read and follow instructions on label carefully. Pesticide labels must provide full instructions on disposal and storage.
- Use up all pesticides according to label instructions.
- Triple rinse containers when empty.
- Return cleaned containers to dealer.

Corporate Liability

The major federal laws on the environment have been briefly described. Let's consider those persons who are covered by these laws. What are the potential penalties for these persons when they are found to have violated the laws?

In general, responsibility for compliance with the environmental laws is imposed on any owner, operator, or person in charge of a facility. A person includes corporations, the corporate officer or officers with the responsibility and authority to achieve compliance with the laws to prevent or correct violations.

For example, if the required action necessary to achieve compliance with the Clean Water Act is the construction of a dike around an oil storage tank on railroad property, there are several "persons" under the law who would be responsible for compliance. The first would be the corporation itself which would be responsible for compliance. Second, the members of management with budgetary, operating, or maintenance authority would be responsible for compliance. Lastly, persons responsible for constructing and maintaining the dike have compliance responsibilities.

Under other circumstances, however, the responsible person may be someone who works at the facility or jobsite on a daily basis. For example, an employee who observes a spill of oil or other hazardous substances from a tank car or a storage tank is required to notify his supervisor so that the necessary notification of government agencies can be made. The employee who first observed the spill would be a responsible person in this instance.

Another instance is where an employee is responsible for discharge sampling procedures pursuant to permits issued under the Clean Water Act. For purposes of taking and submitting the samples, the environmental agency likely will consider the employee at the facility and the railroad company as the responsible persons under law.

Enforcement

Turning to the subject of enforcement, various enforcement mechanisms are available to the government for use against any person or company charged with violating the environmental laws and regulations. Both civil and criminal remedies are available. From a civil standpoint, the EPA or state agency can issue cease and desist orders, administrative consent orders, or they can institute a law suit for an injunction or damages against the alleged violator. The primary purpose of these actions is to seek to have the defendant abate the pollution problem. Lump sum civil penalties of up to $50,000 are possible and in addition, a person can be fined up to $10,000 for each day that a violation continues. Frequently, the agency also will seek to recover the cost of any cleanup that was incurred in the abatement of a spill or similar event. Liability in this regard could amount to $50 million.

Although less frequently invoked, the government has criminally prosecuted, on a selective basis, the most flagrant violators of pollution laws, resulting in fines and imprisonment. Under most laws first time violators are subject to fines up to $25,000 and imprisonment for one year or both. Second violations subject the defendant to fines up to $50,000 and imprisonment for two years or both. Under RCRA fines can be up to $250,000 for a person and $1,000,000 for an organization. Criminal enforcement usually is confined to willful, knowing, and malicious acts as opposed to merely negligent or unintentional violations.

Now that you know something about the pollution control laws that affect railroads and that you could be held responsible for pollution incidents, who can provide you with additional information? Your company's manager of environmental control knows the environmental rules and the situtations on the railroad to which they apply. He can assist you in solving a pollution control problem. Your company's Law Department can also assist in matters involving regulatory agencies and government inspections.

COMMITTEE 14—YARDS AND TERMINALS

Report of Subcommittee No. 10

Cooperate with DOT Transportation Systems Center on study of methodical design of classification yards, collaborating as necessary or desirable with other AREA Committees or AAR Units

B.H. Price (Chairman, Subcommittee), M.J. Anderson, R.E. Mingle, P.E. Van Cleve, J.C. Weiser, D.N. Witt.

Your Subcommittee submits this report as information with the recommendation that this assignment be discontinued. (April 29, 1983)

This subcommittee was established in 1977 to participate in a liaison and advisory capacity with USDOT and FRA who were then undertaking the funding for the development of manuals on freight classification yard design and investigating various problem areas related to yard design. This work was primarily done by SRI International (formerly Stanford Research Institute). The following is a summary of the activities and developments resulting from this undertaking:

Publications

Railroad Classification Yard Technology Manual

Volume I — Yard Design Methods (FRA/ORD-81/20.1) March 1981

Volume II — Computer Systems (FRA/ORD-81/20.11) August 1981

Volume III — Freight Car Rollability (FRA/ORD-81/20.111) August 1982

Case Studies

Potomac Yard (RF&P RR — Washington, DC) (FRA/ORD-81/61) August 1981

East Deerfield Yard (B&M RR — Deerfield, MA) February 1980

Elkhart Yard (Conrail-Elkhart, IN) February 1980

Assessment of Car Speed Controls (FRA/ORD-80/90) December 1980

Noise Control (FRA/ORD-81/18) March 1981

Allocating Loss and Damage to the Railroad Transport Cycle (FRA/ORD-81/64) August 1981

Classification Yard Design Workshops

Workshop I — Chicago, IL — October 30-31, 1979 (FRA/ORD 80/17)

Workshop II — St. Louis, MO — May 6-7, 1981 (FRA/ORD 81/41)

Computer Design Program

A description of these design programs may be found in Appendices A, B and C of Volume 1 of the Railroad Classification Yard Technology Manual listed above.

CAPACITY — This is a program to assist the yard designer in estimating the number and lengths of tracks in the receiving, classification and departure yards. (Appendix A)

PROFILE — This is a program designed to aid in selecting the proper hump yard profile from crest to clearance point in a freight classification yard. (Appendix B)

CONFLICT — This is a program to assist the designer in the development of the track layout at the pull-out (or trim) end of a hump classification yard. (Appendix C)

The above reports and programs may be obtained by writing to the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, VA 22161.

All of these assignments have now been concluded and there is no further FRA funding or activity in the field of freight classification yards. Therefore, this assignment is being discontinued.

COMMITTEE 16—ECONOMICS OF PLANT, EQUIPMENT AND OPERATIONS

Report of Subcommittee No. 3

Subcommittee 3 submits, as fulfillment of its assignment, excerpts from Part 1 of the Final Report by the Engineering Panel on Phase 1 of the study undertaken by the Engineering Panel for the Cost Analysis Organization Subcommittee on the Economics of the "80" vs. "100" ton cars, as supplemented by the Panel's November 1982 supplement which includes carweights other than "80" and "100" tons. Both the Final Report and the November 1982 Supplement have been approved and signed by all members of the Engineering Panel. Part II of the report contains biographies of the members of the Panel; descriptions of the analytical models used; assumptions used when running the analytical models; excerpts from "Long Range Effect of the 100 Ton Car on the Track Structure and Their Impact on Maintenance Planning" by R.E. Ahlf; and AAR Report R485, "Effect of Increasing Axle Loads and Rail Fatigue Life" by Allan Zarembski.

The members of the Panel believe that this report reflects the best estimates of the incremental, or additional, damage done by the 263,000 lb. ("100" ton) car over the 220,000 lb. ("80" ton) car on rail, tie, ballast and subgrade elements of the track structure. These factors as well as similar factors included herein for empties and loaded cars of other sizes, will be used in Phase II of the study where additional costs such as car cost, fuel, locomotive and other operating costs, will be combined with MofW costs to determine the range where each car — "80" ton or "100" ton — may be most economical.

FINAL REPORT OF THE ENGINEERING PANEL — (PHASE I) AS SUPPLEMENTED BY THE PANEL — NOVEMBER 1982

Introduction and Purpose

Railroads have been aware for a long time that larger and heavier cars give them more carrying capacity and may reduce train operation costs. This extra carrying capacity has been particularly useful in improving the rail market position in the transportation of bulk commodities. Unfortunately, these heavier cars also cause more damage per gross ton to the track which results in extra maintenance of way expenses for rail, ties, and ballast.

In the February, 1980 issue of *Modern Railroads,* Mr. Robert Ahlf, Chief Operating Planning Officer of the Illinois Central Gulf Railroad, raised the question as to whether bigger was indeed better when it came to car size. Specifically, he asked if the loaded "100" ton 4-axle car (263,000 lbs. gross weight on rail) was less expensive to operate than the "80" ton 4-axle car (220,000 lbs.). Specifically, Ahlf's hypothesis was that under certain conditions of MofW costs, car utilization, fuel costs, and car costs, the "80" ton car did make better economic sense than the "100" ton car.

At an April 1980 meeting of the Cost Analysis Executive Committee in St. Louis, a subcommittee headed by Mr. G.R. Green of the Western Pacific Railroad was formed to review Bob Ahlf's thesis. To accomplish this task, the subcommittee divided its work into two phases:

- Phase I — Determine a series of MofW life factors which assesses the extra damage of the "100" ton car over the "80" ton car with different rail and subgrade conditions.
- Phase II — After relating these life factors to MofW costs, combine them with other relevant costs to determine the ranges, if any, where the "80" ton car is economically superior to the "100" ton car.

It is felt that the combined work of both phases will enable the subcommittee to comment upon Bob Ahlf's thesis.

This report gives the results of Phase I of this study. It is expected that the MofW life factors presented by the Engineering Panel will be used together with the other costs impacting the "80" vs. "100" ton car issue to determine the range where each car would be economically superior.

Methodology

To arrive at a set of life factors to assess the added damage done by the "100" ton car over the

"80" ton car, the Panel focused on three separate parts of the MofW problem. These were:

— Rail Life
— Tie Life
— Ballast and Subgrade Deterioration

It should be also noted that each of these parts are inter-related and the life factors presented in this report are only for a specific set of track conditions.

The specific methodology for determining a life factor can be seen with the help of Exhibit 1.[1] Exhibit 1 is a curve showing the behavior of continuous welded rail on tangent track with normal wheel loading under mixed traffic in millions of gross tons — MGT's. Note that for traffic with no rail cars whose static axle load is heavier than 28,000 lbs. — or 28 kips — the normal life is governed by the curve marked "wear life". We see that this curve corresponds to the replacement of rail due to excessive head wear.

What Exhibit 1 implies is that for traffic whose wheel loadings were less than 28 kips (the "80" ton car and less), the major contributing factor to replacing rail would usually be head wear. However, for wheel loadings in excess of 28 kips (the normal "100" ton car, for example, at 33 kips) the factor determining the condemnation/replacement of the continuous welded rail on tangent track is no longer rail wear, but fatigue.

The increased rate of defect formation and growth due to fatigue has a rather dramatic impact on rail life. As seen in Exhibit 1 for example, above 28 kips — or for cars heavier than the "80" ton — tangent rail life is dominated by fatigue and its usual life is substantially reduced when contrasted to life with cars less than "80" tons.

It was this reduction in rail life which was used to develop the relative wear/life factors. For example, if the nominal rail life with the "80" ton car is 1,000 MGT and the "100" ton car is 500 MGT, then the life factor on a per ton basis for the "100" ton car would be twice as large as an "80" ton car. The relative factor, F, in this example would be:

$$F = \frac{\text{Life of rail with ``80'' ton car}}{\text{Life of rail with ``100'' ton car}}$$

$$\frac{1{,}000}{500} = 2.0$$

Therefore, to arrive at an incremental life factor for the "100" ton versus the "80" ton car for rail, ties, and ballast and subgrade, expected MGT lives from various analytical models were compared.[2]

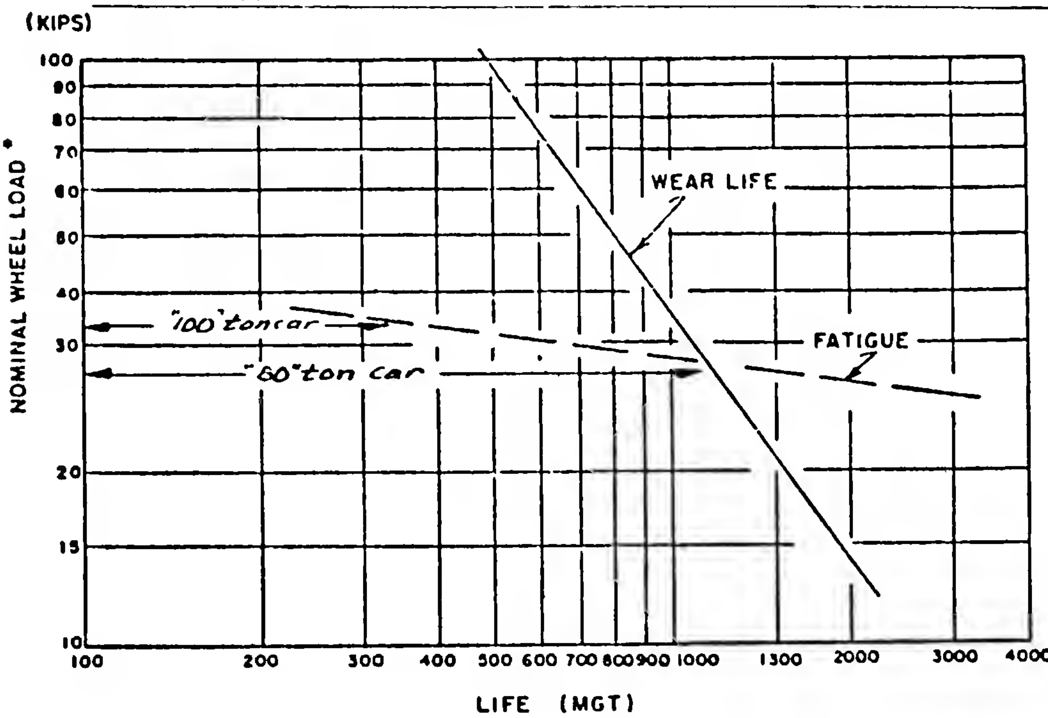
[1] On the onset it became obvious to the Panel that more was known about the effects of heavy loads on the rail itself than on ties or subgrade and ballast. Much basic research is still to be done on ties and ballast.

[2] In the case of ballast and subgrade, the results of the Illi-Track model did not express lives in MGT as did the other models. Instead the methodology was modified slightly to use the ratios of the stresses, strains, and displacements from the respective cars to make estimates of the incremental damage.

EXHIBIT 1

METHODOLOGY (CONT'D)

RAIL LIFE IS REDUCED FOR WHEEL LOADINGS ABOVE THE "80" TON CAR FOR MIXED FREIGHT ON CONTINUOUS WELDED RAIL OVER TANGENT TRACK



SOURCE: Allan M. Zarembski: "Effects of Rail Section and Traffic on Rail Fatigue Life", AAR technical report R-391.

* Largest static wheel loading in thousand pound force -- KIPS.

The models used in this study were:

RAIL	— AAR Rail Fatigue Model
	— Chessie Rail Model
	— CIGGT Rail Life Model
TIES	— Chessie Tie Model
BALLAST & SUBGRADE	— University of Illinois Illi-Track Model
	— Chessie Subgrade Model

To determine their impact upon track life, wide ranges of rail, ties, subgrade and operating conditions were considered. Specifications of the cars and rails used in this study are as follows:

Specifications

— Weight of Car	"100" ton (4-axle 263,000 lbs. gross loaded weight)
	"80" ton (4-axle 220,000 lbs. gross loaded weight)
— Weight of Rail	132# (heavy), 115# (medium), 90# (light)
— Type of Rail	Continuous Welded

— Degree of curvature	0 — 2 (tangent) 2 — 5 5 — 8 8 — 12
— Superelevation	No more than ± 2″ unbalanced
— Subgrade and Ballast	Various combinations
— Speed	Various — usually 40-60 mph
— Impact Factor	AREA Factors or measured load spectra
— Types of Ties	Wood
— Type of Rail Deterioration	Fatigue on tangent track — Wear on curves
— Wheel Size	36″ for "100" ton car 33″ for "80" ton car
— Rail and Wheel Considerations	Various — usually worn wheels on worn rails; std. carbon rail with no curve lubrication.

Results (Life Factors for Rail, Ties, and Ballast)

As mentioned in the Introduction, the Engineering Panel has been asked to estimate the relative lives of track components under different car weights. The factors presented here do not take into account the impact of new truck design, increased wheel diameter, new grinding techniques, and other factors involving technological change. This study presents results based on standard rail and for typical cars in use today.

The best estimates of the MofW life factors are as follows:

— Rail Life	— Exhibit A
— Ties Life	— Exhibit B
— Ballast and Subgrade Deterioration	— Exhibit C

Care must be exercised when using the life factors presented in Exhibits A, B, and C. Detailed track segment analysis as to the MofW damage and costs are obviously superior to the "average" factors presented here. However, the Engineering Panel feels these factors do represent a judgment on the average damage done by different cars on track when site and traffic specific information is not available.

The tables show that "100" ton — or 263,000 lb. — cars shorten rail life to a significant extent. They will also increase some other maintenance of way costs. In all cases, the track life loss per ton is greater for "100" ton cars than for "80" ton cars.

The Engineering Panel wishes to point out certain caveats associated with application of these life factors.

— *Individual movements should be analyzed in detail to determine whether they are economical or not.* For example, if a proposed mine had a total lifetime production of 150 million tons, the fact that rail life on a new line to serve it would be reduced from say 500 million to 250 million gross tons with the "100" ton car, could have little or no economic effect other than the residual value of rail.

— *The Engineering Panel does not state which car may have the optimum gross weight from an overall economic standpoint.* 263,000 lb. and heavier cars can be handled as long as the track is maintained in an adequate condition for them at the relevant operation speeds.

Exhibit A

Life Factors For: Rail

Gross Weight On Rail	Tangent			Curves		
	132#	115#	90#	2-5°	5-8°	8-12°
315,000 lbs.	2.2-3.3	2.5-4.0		4.0-6.0	7.-9.	9.-11.0
263,000 lbs.	1.5-2.1	1.8-2.4	2.3-3.0	3.0-4.1	5.0-6.2	8.0-9.0
220,000 lbs.	1.0	1.2	1.5	2.0-2.5	3.5-4.0	5.5-6.0
177,000 lbs.	0.7-1.0	0.8-1.1	1.0-1.3	1.6-2.2	2.5-3.5	3.8-5.0
60,000 lbs.	0.5-0.8*	0.6-0.9*	0.7-1.2*	1.4-1.8	2.0-3.0	3.0-4.5

*These factors do not take into account severe "hunting."

Exhibit B

Life Factors For: Ties

Gross Weight On Rail	Tangent 132#	Tangent 115#	Tangent 90#	Curves 2-5°	Curves 5-8°	Curves 8-12°
315,000 lbs.	1.4-2.1	1.5-2.2	1.5-2.3	1.7-2.4	1.8-2.7	2.0-2.9
263,000 lbs.	1.0-1.4	1.1-1.5	1.4-2.0	1.2-1.5	1.2-1.5	1.3-1.5
220,000 lbs.	1.0	1.0	1.2	1.1	1.1	1.2
177,000 lbs.	0.6-0.9	0.7-1.0	0.7-1.0	0.7-1.0	0.7-1.1	0.7-1.1
60,000 lbs.	0.6-0.9	0.6-0.9	0.7-1.0	0.7-1.0	0.7-1.0	0.7-1.0

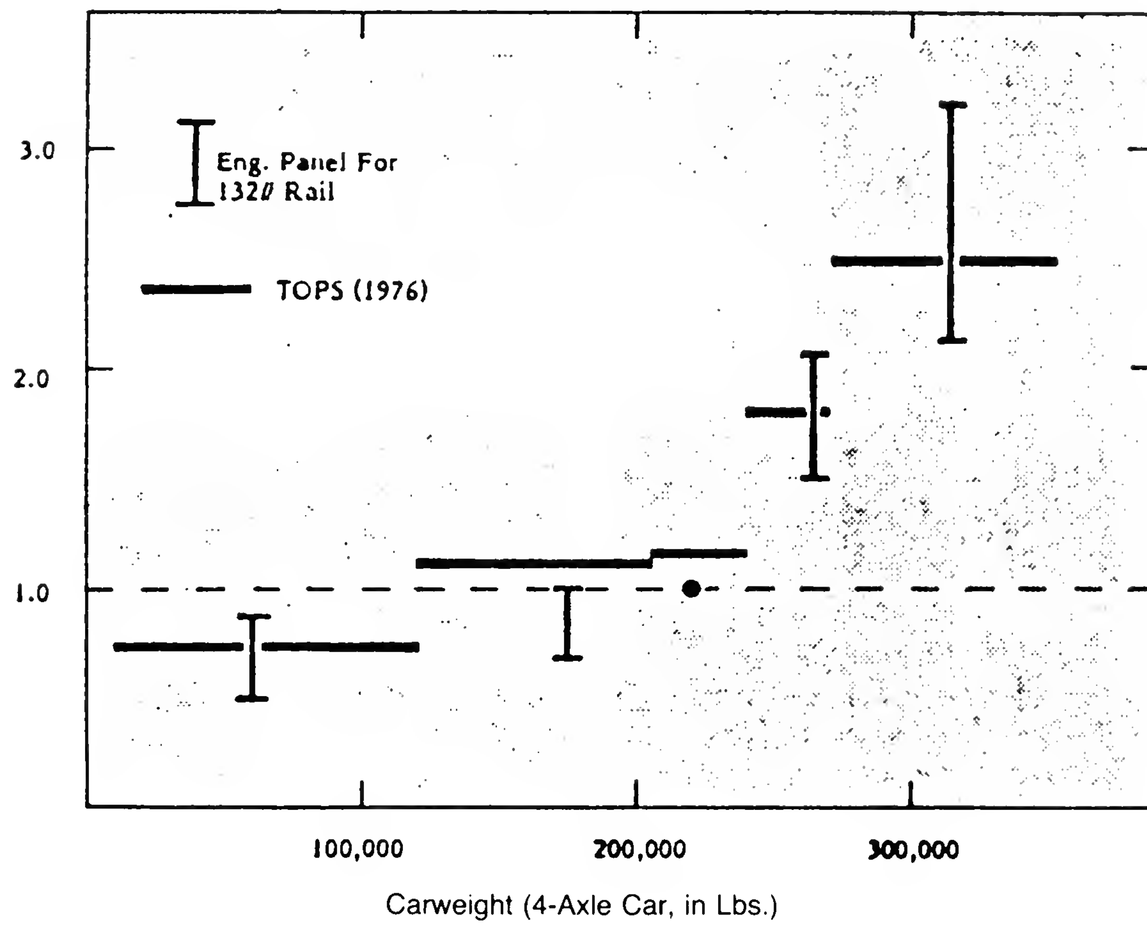
Exhibit C

Life Factors For: Ballast & Subgrade

Gross Weight On Rail	Tangent			Curves		
	132#	115#	90#	2-5°	5-8°	8-12°
315,000 lbs.	1.1-1.9	1.2-2.0	1.3-2.2	1.4-2.4	1.5-2.6	1.7-2.8
263,000 lbs.	1.0-1.4	1.1-1.6	1.2-2.0	1.2-1.9	1.3-2.1	1.4-2.3
220,000 lbs.	1.0	1.1	1.2	1.2	1.2	1.3
177,000 lbs.	0.7-1.0	0.7-1.1	0.8-1.2	0.8-1.1	0.8-1.2	0.9-1.3
60,000 lbs.	0.7-1.0	0.7-1.0	0.8-1.1	0.8-1.1	0.8-1.2	0.8-1.2

It is also possible to compare the Engineering Panel results with those from previous studies. As seen below the Engineering Panel's life factors are compared with those results from the TOPS study performed in 1976 for the Southern Pacific Railroad. In the TOPS study, SP MofW personnel were asked about the relative damage done by different cars on rail. These results can be thought of as a composite of their MofW judgments and do not rely on any specific analytical model. It should be noted that the TOPS results do agree quite well with the results of the Engineering Panel presented here.

Comparison Of Engineering Panel Results With The TOPS Study



RESOLUTION OF THE GENERAL COMMITTEE OF THE ENGINEERING DIVISION OF THE AAR ON THE MAINTENANCE OF WAY COSTS

The General Committee of the Engineering Division of the AAR, meeting on March 22, 1981, and in response to the Cost Analysis Organization of the Economics and Finance Department of the AAR, offers the opinion below. This opinion is based on the practical experience, with presently in-use equipment, of those on the Committee. It in no way implies that a 263,000 lb. or heavier car may not be an optimum gross weight on four axles from an overall railroad standpoint. The optimum axle load for any particular situation is subject to the analysis of the individual details of that situation. Much heavier cars than 263,000 lb. on four axles have been and are safely operated under various conditions. As more physical data are developed, these statements may change.

Keeping in mind the above statements, the following is offered as a general rule of thumb regarding maintenance of way costs alone.

> The life of continuous welded rail on tangent track measured in MGT under 263,000 lb. cars is approximately ⅝ of rail life under 220,000 lb. cars. The other variable maintenance of way costs, per MGT, will be increased 20% for 263,000 lb. cars vs. 220,000 lb. cars on heavy duty track in good condition. This percentage will increase as the condition of a given track decreases, and on poor track, the cost of maintenance may be several times higher for 263,000 lb. cars than for 220,000 lb. cars.

It should be noted that this resolution corroborates the results expressed in Exhibits A, B, and C above.

Conclusions of Phase I

— Rail life is substantially reduced for wheel loadings above the "80" ton — or 220,000 lbs. — car. Rail life in this region is influenced by fatigue as well as wear.

— Best results to date show that the loaded "100" ton car — 263,000 lbs. gross weight on rail — significantly shortens rail life and increases MofW costs on a gross ton basis. This weight on rail also affects tie, ballast and subgrade and will increase these MofW costs per gross ton.

— Individual movements should be analyzed in detail to determine whether they are truly economical. Care must be taken to recognize all relevant costs before a final decision is made on the economics of "80" vs. "100" ton cars.

Background on the Engineering Panel

The Panel held a number of meetings since its formation and between formal meetings discussed problems by correspondence and telephone. In order for the Panel to arrive at the relative life factors, it reviewed results of several rail, tie, subgrade and ballast deterioration investigations and relied upon their combined MofW experience. This combined theory/experience approach is the basis of the estimates made of the incremental damage done by a "100" ton over the "80" ton car with various track conditions.

To accomplish Phase I — the determination of a series of MofW life factors to assess the extra damage done by the "100" ton car over the "80" ton car — an Engineering Panel was formed in June, 1980. The members are[1]:

Prof. William Hay	University of Illinois (Professor Emeritus)
Tom Hutcheson	Consultant (SCL-Retired, Chief Engineer)
Imre Reiner	Chessie System
Lou Cerny	AREA and Engineering Division — AAR
Allan Zarembski	AAR
Bill Lewis (Chairman)	AAR

Requests for nominees to serve on the Engineering Panel were sent to members of the CAO in May of 1980. It should be noted that the Engineering Panel has members from the roads and AREA, as well as having representatives who are involved with "state-of-the-art" track modeling.

[1] Appendix B contains biographies of each member of the Engineering Panel.

COMMITTEE 34—SCALES

The Committee has recommended that the following material be added to the AAR Scale Handbook.

3.0 SPECIFICATIONS FOR THE DESIGN AND INSTALLATION OF LOW PROFILE AND PITLESS RAILWAY TRACK SCALES.

3.1 GENERAL

The following specifications apply to railway track scales that are installed directly on ballast, in shallow pits or on concrete slabs. This section is to be used in conjunction with Part 1 and 2 of this handbook.

NOTE: A shallow pit scale is construed as one that cannot be serviced from below.

3.2 DESCRIPTION

As a general rule, pitless and low profile railroad track scales are manufactured in modules. A single or multiple module may be used for motion weighing.

3.3 CAPACITIES

The multiple module scale, including those with spaces between modules, shall be treated as a single scale.

3.4 BALLAST

Size and type of ballast to be specified by the Chief Engineer of the serving railroad. Unprepared ballast will not be used for railroad track scales.

3.5 SOIL BEARING CAPACITY FOR BALLAST SUPPORTED SCALES

3.5.1 It is the objective to prevent the accumulation of water or other foreign material in the scale subgrade or the scale structure which could degrade the vertical relationship of the scale elements, lead to uneven settlement under traffic, or degrade function and performance.

3.5.2 If gound water is encountered at the proposed scale site and the subgrade is saturated, alternate scale locations should be investigated. If alternate locations are not available, manufacturer's specifications must be followed with the supervision of a competent Soils Engineer. Solutions to soil bearing problems or saturated areas may require overexcavation and emplacement of engineering fabrics, the use of an engineered aggregate base as backfill, the design and installation of a drainage system for the entire scale site or a combination of all three. In unsuitable soils as described above, it is mandatory that the entire process be supervised by a competent Soils and/or Foundation Engineer.

3.6 APPROACHES FOR BALLAST SUPPORTED SCALES

Approaches shall be of the lengths required in Part 2 and the foundation requirements shall be the same as required for the scale.

3.7 SCALE STABILIZATION FOR BALLAST SUPPORTED SCALES

Adequate means shall be provided to ensure scale stabilization, effectively tieing the scale to the ballast or foundation elements with suitable restraining devices.

3.8 MULTIPLE MODULE SCALE INSTALLATIONS

The total scale installation, and its approaches, shall be installed and maintained in horizontal and vertical alignment.

3.9 WEATHER PROTECTION

In regions with snow and ice, means shall be provided so that snow and ice will not accumulate under the weigh bridge and around the load cells.

MEMOIR

Donald L. Nord
1925-1983

Donald L. Nord, Staff Engineer-Structures, Illinois Central Gulf Railroad, died in a hospital near his home in Park Forest, Illinois on May 17, 1983.

Mr. Nord was born March 31, 1925 in Minneapolis, Minnesota. He received his B.S. in Civil Engineering from the University of Wisconsin in Madison in 1948 and his M.S. in Civil Engineering from the same institution in 1950. While pursuing his M.S. degree, he also worked as an assistant instructor and instructor in Civil Engineering at the University of Wisconsin. Mr. Nord's higher education was interrupted from 1944 to 1946 by military service in the Army of the United States.

Don Nord started his engineering career as a Junior Engineering Aide in the Bridge Department of the Illinois Central Railroad in Chicago on February 1, 1950 and advanced through that organization and the merged Illinois Central Gulf Railroad until his death, holding position of Designer, Drainage Engineer, Chief Designer, Engineer of Design, Assistant to Engineer of Bridges and Staff Engineer-Structures. In his final assignment he was responsible for the management of all bridge-related projects involving outside agencies such as the U.S. Army Corps of Engineers, U.S. Coast Guard, State Departments of Transportation, and others.

Mr. Nord was a Registered Professional Engineer in Illinois and a member of the American Railway Engineering Association and the American Railway Bridge and Building Association. In the A.R.E.A. he has been a member of Committee 15, Steel Structures, since 1959 and had served as its Chairman in 1968-1970. Since his chairmanship, he continued to actively progress the goals of the Committee, and served with distinction as Chairman of its Subcommittee on Revision of Manual.

In addition to his railroad activities, Mr. Nord was an active member and supporter of the Church of the Holy Family in Park Forest, Illinois.

Mr. Nord is survived by his wife, Naomi, "NIMI" of 405 Wilshire Street, Park Forest, Illinois 60466, by two sons, Thomas C. and Steven J., and three grandchildren.

Donald Nord was a highly skilled and professional bridge engineer who could be depended upon to promptly complete any and all assignments, both of Committee 15 and his Railroad. He is personally and professionally missed by all members of the engineering profession with whom he had contact.

John E. Barrett
Dan S. Bechly

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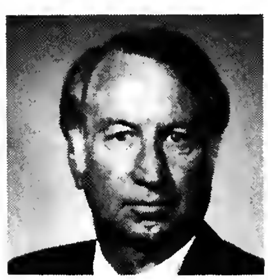

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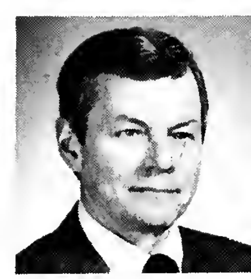

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American Railway Engineering Association

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CONTENTS

Cover Photo: Eastbound commuter train heads for Montreal on Canadian Pacific mainline west of Dorval, Quebec. Canadian National mainline is at left.

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West of Montreal

There are many places in the railway network of the North American continent where a large amount of railroad activity is in view at one point. Such a place is a 10-mile stretch in Canada east of Montreal, Quebec between Vaudrevil and Dorval, where the double-track main lines of the Canadian National and Canadian Pacific run alongside each other. In addition to the heavy freight traffic on both lines, the Canadian Pacific tracks handle commuter trains of the M.U.C.T.C. (see front cover) and the Canadian National tracks handle VIA Canada Rail passenger trains from and to Toronto, Ottowa, and as far west as Vancouver, British Columbia. (see next two pages)

The photo above shows an eastbound Canadian National freight on this section of track, while that below shows an eastbound commuter train on the Canadian Pacific track with the Canadian National line on the left.

Above: Canadian Pacific eastbound container train west of Montreal, Quebec.

Below: VIA Rail Canada train with conventional equipment heads west on Canadian National track.

All photos in this article were taken July 1983.

Above: VIA Canada LRC train (Light, Rapid, Comfortable) on Canadian National track west of Montreal. These trains are allowed a maximum speed of 95 m.p.h.

Below: Westbound Canadian Pacific freight at same point as above.

Update on Rail Specifications on CN Rail

A. W. Worth*

1. THE GENERAL NORTH AMERICAN PROBLEM

Fifteen years ago the problems with rail in North America generally seemed to have been solved. Controlled cooling of rail was introduced in the 1930's and was steadily reducing the incidence of transverse defects. Continuous welded rail had been in use for ten years or more and brought dramatic reductions in problems with rail end failures, batter and surfacing. Modern RE rail sections with generous fillet radii and more scientifically designed head contours were steadily reducing incidence of head and web cracks, base and web cracks, and shell. For especially heavy service in sharp curves, some railroads installed high silicon or heat treated rail and reported excellent results.

Into this stable world the 100-ton capacity freight car dropped with shattering impact. These cars proliferated in unit trains of 100 ton cars of 12000 or more trailing tons. Many railroads found horrendous problems trying to keep up with the rate of track destruction. When the dust settled we found the life of the ordinary carbon steel rail of the day in curves was not much more than half what we had expected a few years before. The problem was not just conventional head wear. The rail rapidly crushed and corrugated, and where the rail did not fail for these reasons it eventually began to shell severely.

The cause of the problem was that the stresses in the wheel-rail contact area had gone over the threshhold of capacity of the rail we were using, which had a hardness of only 240 to 280 Brinell as received from the mill and a yield strength of 60,000 to 65,000 psi.

This problem was predicted by Mr. C. J. Code, former Chief Engineer of the Pennsylvania Railroad, as long ago as 1959, when in a letter to the Mechanical Division of the AAR he recommended wheel loads on 36 inch wheels not be permitted to exceed 810 lbs. per inch of wheel diameter. However, for a variety of reasons not related to civil engineering the AAR did not adopt this recommendation. Being unsuccessful in limiting stresses to what existing rail steels could take, we found ourselves forced in the other direction of trying to find a steel strong enough to take the loads imposed by the new cars.

From early trials of heat-treated rail we found that just making the rail harder was not enough. It also had to be made cleaner. Hertzian stresses under the wheel-rail contact area had increased. As measures were taken to reduce the rate of loss of rail head metal from wear and crushing, rails began to fail through shell rather than through loss of head dimension. Work by Dr. Marich in Australia suggests that shell is the result of Hertzian stresses in the fatigue range. If the rail wears or crushes rapidly, the zone of maximum stresses keeps moving and not enough fatigue cycles will accumulate at any one point to cause shell. If the rail is made harder, so the maximum stress zone doesn't move, shell may result.

Examination of many shelled rails in our laboratories showed at least a qualitative correlation between non-metallics and shell in the rail, which suggested that non-metallics probably were a factor in reducing the fatigue life of the steel and precipitating shell. The Japanese and Australians have maintained such to be the case for many years. It is evident from many presentations made by Roger Steele of the AAR over the last year that the Metallurgical Section of the AAR technical group now has come to the same conclusion.

The question of microcleanliness has been particularly difficult for steel mills to address till recently because it seemed to be an intrinsic property of a particular plant. It could be improved marginally by taking extra care, but quantum improvements could not be made without a "root-and-branch" tearing out of much older basic steel production capacity.

North American mills have long been at a disadvantage in this respect because of the age of their plant. In France or Japan a mill more than 10 or 15 years old is considered obsolete. In North

* System Engineer Standards, CN Rail

America, some parts of a mill may be 40 to 80 years old. Process control in such mills intrinsically is more difficult.

We may now have a way out of this dilemma. Recently U.S. mills have introduced what they term clean-steel practice, which consists of introducing an artificial slag-producing flux into the molten steel (generally in the ladle) and blowing argon through it. The finely divided non-metallics in the melt adhere to the flux and are skimmed off. This practice is too new to show positive results in service, and to date it has not been introduced in Canada, but consensus is its general adoption is inevitable. If successful it will be a significant advance.

Meanwhile, beginning in the 1970's, a search has been going on for a harder and stronger rail that could be made with existing equipment at a price the customers were willing to pay. Beginning with a few limited tests of simple alloys, research has broadened and expanded into a simultaneous coordinated effort that seems to involve all the railroads and all the rail mills in the industrialized world.

2. SPECIFIC CANADIAN PROBLEMS

In Canada these general problems are compounded by some that are peculiarly our own. The most obvious is climate. Where rail temperatures dip below minus 40 degrees, we can not tolerate compositions of hard alloy steel that show too much loss of notch toughness at such temperatures. Continuous welded rail is under a high shrinkage stress at low temperatures and the combination of a frozen roadbed, flat or shelled wheels and the general tendency of notch toughness to drop with temperature puts rails to a very severe test.

Our heaviest hauls, both in terms of total tonnage and of axle load, are in British Columbia and western Alberta, through a territory of mountains and canyons with particularly severe curvature. The track is congested and generally inaccessible, so it is important that rail life be as long as possible.

On some of these curves, in the early 1970's, ordinary 39 foot carbon steel rail lasted as little as 18 months, even with transposing. Admittedly the lubrication was imperfect, the value of lubrication not being fully appreciated at the time. We knew a variety of rails with better wear properties was on the market, but we needed to know which of them was most cost effective. This led to detailed examination of results of in-track tests we had begun in the 1960's of four types of improved rail: Algoma Chrome, Sydney Chrome Vanadium, U.S. Steel Curvemaster and Bethlehem Fully Heat Treated Rail.

The Algoma rail was softer than we really wanted. The U.S. Steel and Bethlehem rails were not available on the market in Canada in sufficient quantity to be a practical alternative. Sydney did not want to make chrome vanadium rail any more, as their metallurgists thought they had come up with something better: Chrome Columbium.

Sydney rolled a small tonnage of 132 lb. Chrome Columbium for us and we put it in curves on our B.C. south line. Everything went well. After two years in track it looked as if the rail was going to last 3.5 times as long as carbon steel. Sydney quoted us an attractive price, and in 1978 we placed our first large order for Chrome Columbium rail, for delivery for the 1979 rail program.

As so often happens, the success achieved with the first small test lot did not repeat itself in full-scale production. The alloy proved extremely difficult to roll. About half the production failed to get out of the mill at all, and much of the rail received from the first year's rollings still was found to have laps, scabs and seams. After making about 5000 tons of finished rail Sydney ceased production; they were just losing too much money.

Another attempt in 1981 to roll chrome columbium again lost money, and culminated in the spring of 1982 when 6400 rails had to be shipped back from our B.C. South Line for re-inspection because base seams were causing them to fail in track.

The base seam problem has since been brought under control by magnetic particle testing and ultrasonic inspection, but meanwhile another problem has become evident: shatter crack.

Ever since alloy rails began to be used in quantity, we have noticed occasional flurries of "infant mortality" failures. In the Chrome Columbium rail these often appeared as VSH's and initially were blamed on segregation. In the Algoma Chrome, being near the ends, they were confused with weld failures. However, on laboratory examination the real cause was found to be shatter crack from hydrogen. We have come to believe that when chromium levels in the steel exceed 1%, as they do in the Sydney and Algoma alloys, the conventional tank cooling process may not always be able to cope with all the hydrogen in the hot steel if the percentage of hydrogen in the melt is at the top end of the range.

These difficulties, too, can be overcome by additional expenditures on vacuum degassing equipment or additional quality control checks, but these add further to cost and difficulty of production. Also, the two inherent problems with use of any conventionally cooled alloy steel rail can not be made to go away. One is that to guarantee 100,000 psi yield in 100% of the product seems to be impossible, as statistical scatter will make some of the steel so hard it is excessively brittle. The other problem is that, unless molybdenum is used, alloy steel rails approaching 100,000 psi yield become progressively slower and more tricky to weld. Alloys containing molybdenum have a variety of quality control problems all their own, inconsistent hardness being one and "ghost lines" at magnaflux inspection of hot butt welds being another. It is progressively becoming more evident that the route of using conventional rail cooling with relatively high alloy contents, on which we all set out so bravely in the 1970's, is a blind alley.

3. COOPERATION WITH INDUSTRY

We have said a certain amount about things going wrong with Sydney and Algoma rail. In fairness, it must be said that the reason we have concentrated on these two mills is that we have much more experience with their product than we have with that of mills from outside Canada. The general experience of all mills world-wide seems to have been the same. They were trying to turn out a product whose properties they did not fully understand with the equipment at their disposal at a competitive price. Rather, it just points out that when a company attempts to get ahead of the competition and to 'boldly go where no man has gone before', some things can go spectacularly wrong very fast.

In recent years, in our discussions with various steelmakers we have tried to provoke a cross-fertilization of ideas for the good of all. It has often seemed to us that the steel industry has suffered from the 'not-invented-here' syndrome, or perhaps pure cussedness. If Algoma made a straight chrome, then Sydney would make a chrome vanadium or chrome columbium, CF&I a chrome molybdenum, Sacilor a chrome-silicon-vanadium, British Steel a chrome-manganese, U.S. Steel a head hardened rail, and Bethlehem a fully heat treated rail. This causes no end of problems at welding plants and at frog and switch manufacturing plants, as well as to the track maintenance people who have to maintain the material afterward, when for reasons of availability, price or quality control we have to change suppliers. Perhaps as we evolve from the era of high alloy contents and conventional cooling into the era of more use of heat treated rail this proliferation of exotic alloys will slow down. It is all very well to 'let 1000 flowers bloom', but reason has to prevail sometime.

4. ALLOYS WE HAVE TRIED; RESULTS

Our first large purchases of alloy rail were Chrome Columbium from Sydney Steel. This probably was a harder and more wear resistant alloy than any we have tried since, but it had so many production problems that Sydney gave it up in the spring of 1982.

Initial wear tests with Chrome Columbium rail suggested a life perhaps 3.5 times that of carbon steel rail, but with more traffic (and more rail grinding) this estimate has been revised progressively downward. After several more years' service it now appears likely to last about 2.5 times as long as carbon steel. This downward shift in projected life expectancy with traffic remains difficult to explain, and initially caused us some embarrassment, until the same phenomenon showed up in the alloy rails in the FAST experiment.

Algoma Chrome is a straight chromium. When first introduced its physical properties were governed by chemistry only, so we found yield strengths anywhere from 88,000 to 100,000 psi. In co-operation with us, Algoma has now brought in quality control methods that have reduced this spread considerably, so they now consistently produce a product that is 95% over 95,000 psi, and nothing under 93,000 psi yield. This seems to be the limit achievable with a straight chromium alloy. This strength being less than that of Chrome columbium, we do not expect it to last more than twice as long as carbon steel.

Sacilor Chrome-Silicon-Vanadium is intermediate in properties between Chrome Columbium and Algoma Chrome, and its life likewise probably will be intermediate, perhaps 2.25 times that of carbon steel.

CF&I Cromorail, of which we bought some 4000 tons in 1981, proved to be somewhat inconsistent in yield point and hardness, apparently due to differences in cooling rates on the hot beds. The harder readings approach those of Chrome Columbium, but the softer are more like Algoma Chrome. Because these hardnesses are so mixed, the ends of individual rails being harder than the middle, we do not expect these rails to last more than twice the life of carbon steel. CF&I is aware of this problem and has stopped offering large orders to us till they can correct it.

Nippon Steel and NKK make two types of head hardened rail. The first to be introduced to the market was a standard carbon steel rail, head hardened, of which we bought 5500 tons in 1982. This rail has not been in track long enough on our lines to get any projection of life expectancy, but based on tests at FAST we would expect it to last about 2.5 times as long as carbon steel.

The latest type of rail introduced by Nippon Steel and NKK is already being called 'super rail'. It is a low chromium, high silicon, head hardened alloy with properties superior to anything else we have seen. We expect our next order for heavy tonnage curves to be for this type of rail.

We might add that in the 5500 tons of Nippon Steel and NKK rail we bought in 1982 we have had absolutely no failures reported whatever. There have been some instances of soft spots and minor batter at the gauge corner adjacent to welds. The extent to which these were due to unfamiliarity of welding employees with the new air quench equipment is not known. There also have been instances of gauge corner shell. The Nippon Steel experts had not seen this type of shell before and guessed it might be due to much of the rest of the rail in the line being curve worn, so the wheel flanges had worn to match the worn rail and had a mismatch with new rail. If so, further shell can be avoided by profile grinding. However, we must ensure the local people do not make the same sort of heavy multiple-pass grind on top of the rail head that is used to control corrugation on carbon steel. If they do, the life of the rail will be much reduced.

The wear lives just given apply on curves two degrees and over, on continuous welded rail. On such curves we expect rail to be removed for wear rather than fatigue. We do not attempt to predict what the life of alloy or heat treated rails would be in tangents. We have not been able to develop economics justifying use of other than carbon steel in tangents, and as a result the amount of harder rail installed on tangents that could be used as a data base is negligible.

Comparison of these rates with those from FAST will show they are closer to those from unlubricated rail than those FAST obtained for lubricated rail. Despite much attention to rail lubricators, we never have been able to duplicate the very long lives achieved in lubricated track at FAST.

On the basis of these figures, and taking into consideration all the problems we are continuing to have with alloy steels, the technical superiority of heat treated rails is self-evident. Not only does heat treated rail wear better, but it is easier to weld and does not cause compatibility problems in pairing it with other types of rail. Head hardened rail also is easier to drill, bend and straighten. Blunt drill bits are not as likely to cause bolt hole cracks from martensite formation. There is some difficulty in ensuring proper hardening in the heat affected zones adjacent to welds in head hardened carbon steel rail, but use of low alloy chrome silicon head hardened rail can avoid such problems as well. Except in frog and switch work, where the necessary machining operations remove the head hardened material where it is most needed, the chief roadblocks to an even wider use of heat treated rail at the moment are availability and price.

5. NEW CN RAIL SPECIFICATION

After several years' work and co-operation with steel mills both in Canada and overseas from which we bought rail, in April 1983 we published our Specification 12-4, covering premium steel rail. This sets out chemistry and physicals required for both chrome alloy and heat-treated rail, together with quality control measures we require.

In the case of alloy rail, the standard composition we prescribe is essentially a straight chrome rail, similar to that put out by Algoma Steel, 1.05%—1.35% chromium. At the time of writing, we thought that the prescribed physicals could possibly be met by a straight chrome, but as more and better test data became available it has become evident that to get 100,000 psi yield vanadium must be added. In any event, we require the carbon equivalent to be held between 1.06 and 1.20 to preserve weldability.

Carbon equivalent is a formula useful for roughly equating the aggregate effects on physicals and weldability of an alloy from various admixtures. We define it as:

$$C.E. = C + \frac{Mn}{5} + \frac{Si}{3} + \frac{Cr}{10} + \frac{V}{1} + \frac{Mo}{1}$$

For example, a steel with 0.7% carbon, 0.75% maganese, 0.3% silicon, 1.2% chromium, 0.06% vanadium, would have a carbon equivalent of 0.70 + 0.15 + 0.10 + 0.12 + 0.06 = 1.13 which would meet specification.

Alloy rail must show a yield strength of at least 100,000 psi, except that an amount up to 15% of any order may be between 95,000 and 100,000 psi. Heats below 95,000 psi are rejected. Hardness must be between 321 and 388 Brinell, and specified elongation is 9 per cent.

The heat treated rail covered by the specification is a standard carbon steel rail, heat treated. (As yet we do not have a specification for low alloy chrome-silicon head hardened rail.) It is required to show yield strength 114,000 psi and hardness 341 to 388 Brinell, with specified elongation of 10 per cent. The 114,000 psi figure here can not be compared directly to the 100,000 psi figure quoted for alloy rail, because in conformity with industry practice the samples are taken from different places in the rail head.

The full length of our specification covers 22 pages, and obviously can not all be discussed here. Nothing in it is a secret. Anyone interested can discuss it either direct with us or through AREA Committee 4 and the AISI-AREA Joint Contact Committee. It already has been the subject of several discussions at AREA and AAR meetings over the last year.

We really believe that if we can get a head hardened rail like that produced by Nippon Steel or NKK, of consistent hardness, metallurgically clean, straight, in dimensional tolerance, and free of cracks and flaws, we will be ready for the 21st century. The challenge is down to North American industry. Hiding behind tariff barriers will only make things worse in the long run. If we are not to become a second-class backwater we have to make ourselves equal to the best. I believe we can do it. We have to do it. The most important thing is to try.

Major River Crossings on British Columbia Railway's Tumbler Ridge Branch Line

G. W. Taylor [1] and H. S. Kleywegt [2]

INTRODUCTION

The recent completion of the two billion dollar North East Coal Project indicates the importance attached to the vast natural resource wealth of British Columbia. High quality metallurgical coal from two surface mines near the newly created town of Tumbler Ridge in northeastern B.C. is now moving westward by rail to new port facilities near Prince Rupert, B. C. for trans-shipment to the markets of the Pacific Rim. The 460 million dollar, 129 kilometres long Tumbler Ridge Branch Line was constructed by the provincially owned British Columbia Railway Co. to link the coal mines to existing BCR trackage 121 kilometres north of Prince George, B. C. The new rail line, powered by a 50,000 volt overhead contact system, pierces the Rocky Mountains with two major tunnels, 9.0 and 6.5 kilometres long, and has six major and five minor river crossings. Seven million tonnes of coal per year will be shipped west on 98 car unit trains, first by BCR to Prince George and then on to the Ridley Island port by the Canadian National Railway.

PLATE 1 — Parsnip River Bridge

ITEM	PARSNIP	MURRAY	WOLVERINE
TOTAL BRIDGE LENGTH (m)	169	157.	132
SPAN LENGTHS (m)	84,84	20, 9, 45, 9, 45, 9, 20	29, 12, 50, 12, 29
SUPERSTRUCTURE STEEL MASS (TONNES)	720.	530.	470
SUBSTRUCTURE CONCRETE VOLUME (m^3)	935	1250	1230

TABLE 1 — Bridge Statistics

[1] Senior Design Engineer, Ker, Priestman & Associates Ltd., Victoria, B. C.

[2] Resident Engineer, British Columbia Railway

The successful completion of the Tumbler Ridge Branch Line and indeed the entire North East Coal Project was a major technological achievement which demonstrated the skill, ingenuity and dedication of the Province's human resources. The technical and innovative designs and construction techniques are highlighted in this description of the design and construction of the three largest bridges on the new branch line.

The conventionally designed two span, through trusses of the Parsnip River Bridge were cantilevered 84 metres while being launched across the flood-swollen river. The unusual erection method was employed here for only the second time in North America.

A novel and daring design approach for a railway structure was adopted for the Murray and Wolverine River Bridges. A continuous steel box girder featuring an integral ballast trough and steel delta legs was constructed with substantial savings at both bridge sites. The shore-assembled box girders were also erected by launching.

The three bridges described herein were designed and their construction supervised by the consulting engineering firm of Ker, Priestman & Associates Ltd. of Victoria, B. C.. The steelwork erection engineers were Buckland and Taylor Ltd. of North Vancouver, B. C.

BRIDGE SITES

The three bridges comprise the first major river crossing and last two major river crossings on the Tumbler Ridge Branch Line. The first crossing, only 3 km from where the branch line departs from the main track, is the Parsnip River Bridge. Before the rail line reaches its terminus, the line crosses the Wolverine River Bridge and the Murray River Bridge, 16 km and 13 km by rail respectively from the coal load-out facilities.

Good access was provided to all bridge sites during construction. The road into Tumbler Ridge was subsequently upgraded to provincial highway standards following the completion of the rail line.

PLATE 2 — Murray River Bridge

DESIGN LOADING

Live Load

To reflect the nature and magnitude of the unit trains using the branch line, the design live load chosen was Coopers E-80 Single Track loading as defined in the American Railway Engineering Association design code.

Ice

Since Tumbler Ridge is in a continental climate zone and is close to 55° north latitude, both the Murray and Wolverine Rivers are usually frozen for at least five months each year. Whereas the river ice does not reach the large thicknesses common in the Canadian Arctic, the ice loading on river piers in the region is still very substantial.

The ice loading adopted is a pressure of 1400 kPa for an assumed thickness of 600 mm. This ice pressure occurs at breakup under melting conditions when the ice sheet moves in large pieces and is internally sound. The level at which the ice force is applied to the river pier is set by the water level corresponding to the maximum daily flow possible in a "200-year" event at the time of year river breakup occurs.

Temperature and Other Forces

Since this part of the province is subjected to a wide temperature variation, the bridges are designed for a temperature range of 55°C above or below the assumed erection temperature, an important design consideration for both the Murray and Wolverine River Bridges.

The common occurrence of floating debris in the narrower reaches of these rivers is considered in the calculation of stream forces acting on river piers. An allowance is made for an accumulation of logs on the upstream face of the pier under the severest flood conditions.

Earthquake forces do not govern in the design.

PLATE 3 — Wolverine River Bridge

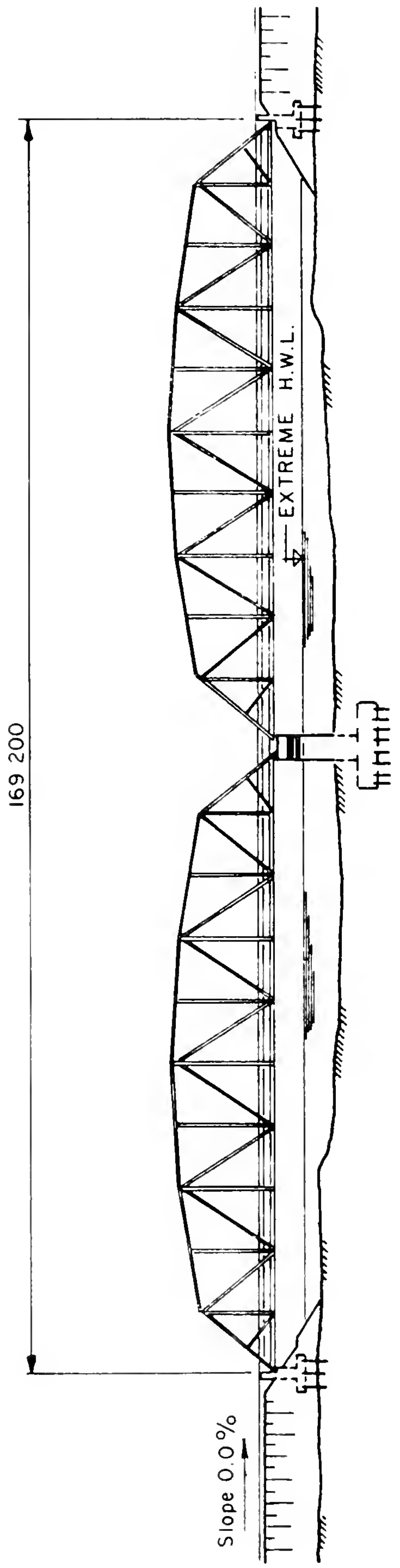

FIG. 1 ELEVATION — PARSNIP RIVER BRIDGE

HYDROLOGY

Parsnip Watershed

The Parsnip River issues from the Parsnip Glacier, an alpine glacier on the western slopes of the Misinchinka Ranges of the Rocky Mountain chain. The river follows a lateral valley from the glacier to the Rocky Mountain Trench where it flows into Williston Lake.

Murray—Wolverine Watershed

The Murray and Wolverine Rivers rise on the eastern slopes of the Hart Ranges of the Rocky Mountain chain. Their basins span a transition zone in which precipitation decreases sharply from high rain and snow accumulations in the Rocky Mountains west of the divide to low values typical of the prairie region of northeastern British Columbia. A considerable snowpack is accumulated annually at the high elevation areas of the basin and consequently every year there is a marked spring "freshet". However, widespread summer rainstorms of several days duration do occur in the region and can cause peak flows greater than the flows caused by snowmelt alone.

The local geology is characteristically sedimentary with river canyons exposing sandstone and shale layers. The drier climate has yielded a moderately dense tree cover of medium height spruce.

Estimation of Design High Water Levels

Determination of the design flood is based upon extrapolation of a statistical plot of peak flows measured on the rivers in the region to yield a "200-year" event. Adoption of the "200-year" flood criterion is common for major highway and railway bridges in British Columbia. Having established the design flood, site-specific estimates of water levels were determined using Manning's formula at three cross-sections in the vicinity of each bridge.

The Parsnip River at the bridge location is flat sloped with an average gradient of 0.1%. Bank-full discharge at the bridge site was determined to be 540 m^3/s and the design flood discharge was set at 1160 m^3/s. The design high water level is approximately two metres above bank-full.

The Murray River has a similar slope to that of the Parsnip River at the bridge site. Having determined the "200-year" flood to be 1830 m^3/s, the design high water level was calculated to be at an elevation approximately three metres above the bank-full stage. Ice scars were noted on the trees lining the banks at close to the design high water level, but these abrasions were probably made by "cake" ice upstream of ice jams that are common on the Murray River.

In contrast, the Wolverine River is a much steeper and faster-flowing waterway than the Murray River, the average slope being 0.5% in the vicinity of the bridge site. Immediately upstream of the bridge site, the river has cut a steep canyon through a sandstone and shale formation, and the river is still confined by the same rock formation on its north bank immediately upstream of the bridge centreline. Just downstream of the bridge crossing, the Wolverine has built a large fan that intrudes on the Murray River floodplain. Along the lower portion of the fan, the Wolverine channel is unstable. Reflecting the high stream velocity, Wolverine bed material is on the average approximately 150 mm in size with occasional large boulders. The design flood for the Wolverine has been established at 890 m^3/s and the corresponding water level is one metre above the south bank.

Scour and Erosion Protection

Because of the high stream velocity and narrower waterway, river piers were not considered for the Wolverine River. However, structural considerations prompted the inclusion of one river pier in both the Parsnip and Murray Rivers where less severe scour effects were anticipated. On the basis of stream velocity and size of bed material, the Murray River was not considered to be subject to further channel degradation. Three metres of bed degradation was assumed for the Parsnip River.

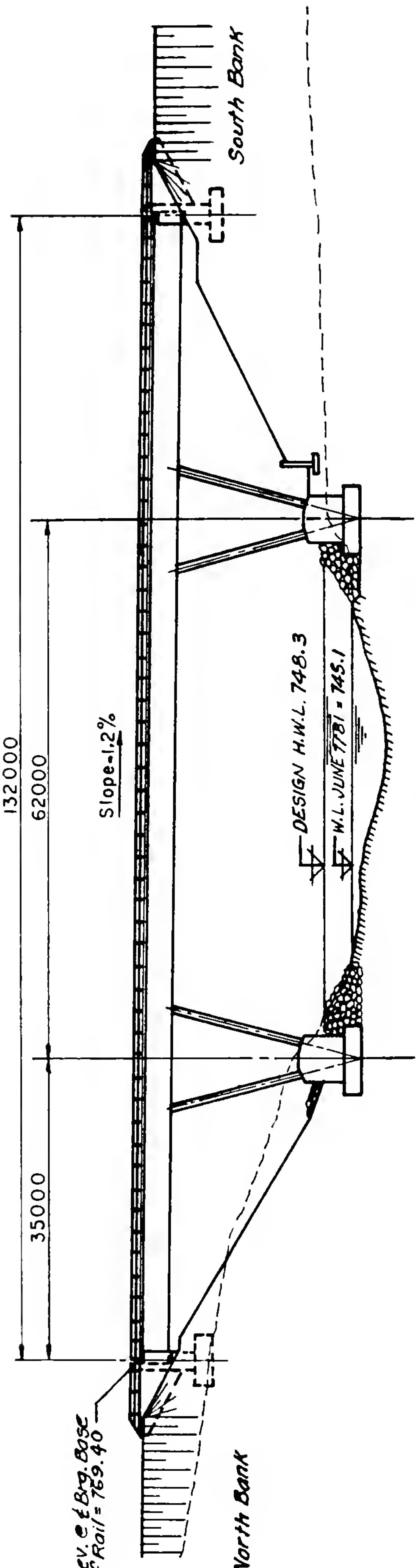

FIG. 2 ELEVATION – WOLVERINE RIVER BRIDGE

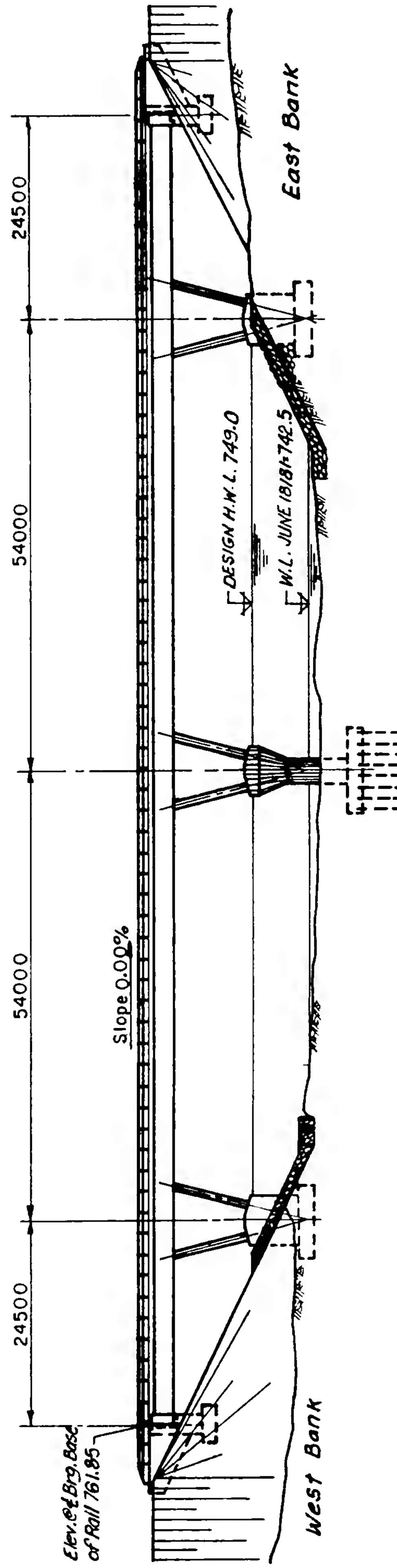

FIG. 3 ELEVATION – MURRAY RIVER BRIDGE

Since both Parsnip and Murray River crossings are skew to the river, river pier shafts were made circular to minimize local scour effects. By keeping the top of the footing below general scour depth, advantage was taken of the size of the footing to minimize the effect of local vortex action. A conservative local scour depth of 4 metres was adopted in the design of both river piers.

Erosion protection is provided on both banks of the Parsnip and Wolverine Rivers. To protect bridge embankments, the rip-rap has an average size of 750 mm. However, the approach end fills of the Murray River Bridge are not as exposed as those of the Parsnip and Wolverine River Bridges and rip-rap has been placed only to protect the bank piers against scour.

DESIGN CONCEPTS

The different bridge site topography and track alignment prompted contrasting design concepts, a more conventional approach for the Parsnip River Bridge and an innovative and unusual approach for both the Murray and Wolverine River Bridges.

Dominant considerations in the formulation of the design concept for most major river crossings are usually economic, topographical, hydrologic, geotechnical and structural in nature. At these bridge crossings, several additional considerations determined the final bridge forms.

To preserve and protect fish life, stringent environmental restrictions apply to all "in-river" construction. The limitation of "in-river" construction to only two months in the summer makes such construction expensive and narrows construction options. Consequently only one river pier was considered for both the Murray and Parsnip crossings.

As both the Murray and Wolverine bridges are in close proximity to the new highway and the Tumbler Ridge townsite, aesthetic bridge forms were of primary importance.

For the Murray and Wolverine River Bridges, economies of scale and a desire to have some similarity of form dictated that design, fabrication, and erection savings could be achieved if the same structural form was utilized for both structures. This, together with the remoteness of the site and the relatively high cost of mobilization suggested a single contract for all substructure construction and a second contract for all steelwork fabrication and erection.

STRUCTURAL SYSTEMS

Parsnip River Bridge Structural System

Since the Parsnip River Bridge is a low-level crossing, the only option was a through truss as illustrated in Plate 1 and Figure 1. The superstructure comprises two ten panel camel-back truss spans with a cast-in-place concrete retainment for a ballasted roadbed.

Murray and Wolverine River Bridges Structural System

In contrast, the Murray and Wolverine crossings are medium level crossings. There is insufficient clearance above the Murray River design high water level for a deck truss to be economical. The height of both crossings permitted the use of steel delta legs on top of the piers to support a single cell steel box girder. By utilizing pairs of delta legs over the piers, the box girder is effectively restrained against rotation over the piers, dramatically reducing deflection, and distributing positive and negative moments equally to generate a highly efficient superstructure. Such a superstructure has 25% less steel than a more conventional truss span. The top flange of the box girder serves a dual function in acting as a containment trough for the ballast as well as the orthotropic top flange for the box girder.

In generating span end fixity, certain delta legs are subject to considerable live load uplift. In addition, the longitudinal restraint of the delta leg rigid frame results in significant leg tensile forces under extreme temperature conditions. To resist these uplift forces, the bases of the delta legs are post-tensioned to the tops of the piers. This post-tensioning also ensures stability under wind.

The structural system adopted for the Wolverine and Murray River Bridges is illustrated in Figures 2 and 3 respectively. The grade of weathering steel used for all bridge superstructures was 350AT Category 2 in accordance with CAN3-G40.21-M81.

SUPERSTRUCTURES

Railbed

The rail line is a single track comprising 57.2 kg/m continuous welded rail spiked to No. 1 timber ties at 500 mm spacing supported on crushed rock ballast. The ballasted decks of all bridges have a minimum ballast thickness under the ties of 200 mm with provision for a future lift of 150 mm.

The design parameters for grades on the branch line have been set at a maximum adverse grade of 1.2% for loaded trains and a 1.5% adverse grade for unloaded trains. Track grade is level for the Parsnip River Bridge and level for most of the length of the Murray River Bridge with the initial segment of a vertical curve at the eastern end of the bridge. The Wolverine River Bridge track has a constant slope of 1.2% adverse to loaded trains approaching from the south.

Parsnip River Bridge Trusses

As noted earlier, each through truss span consists of a ten panel, camel-back Warren truss. All truss chord and web members are welded perforated box members with fully bolted connections. The floorbeam and stringer floor system supports a cast-in-place concrete deck that acts compositely with the stringers.

Box Girders—Murray and Wolverine River Bridges

As noted earlier, the top flange of the box girder acts as the containment trough for the ballasted deck. The principal advantages of a steel rather than a concrete trough are light weight and freedom from curing constraints. The top flange also acts as an integral part of the orthotropic deck. To protect the steel trough against corrosion, the upper surface of the top of the flange and the inside surface of the wing plates that form the sides of the trough are metallized to provide a cathodic protection system. Damage to the metallizing by the ballast is prevented by the placement of asphaltic impregnated ballast panels on top of the metallized coating. To provide a corrosion reserve, the top flange thickness has been increased by 2 mm beyond that required structurally. Since water ponding in the trough presents a serious corrosion hazard, drains are provided every 10 m along the bridge, the drains being positioned outside the girder webs to facilitate maintenance. The top flange is crowned at the bridge centreline to ensure drainage out to the peripheral drains. The wing plates are field bolted to the box girder in short lengths to prevent load sharing with the box top flange.

Considerable structural efficiency is achieved in having the top flange carry load in orthogonal directions. Overall girder moments due to dead load, live load and temperature are resisted by the top flange in the longitudinal direction, whereas local wheel loads are carried through flexure of the top flange in the transverse direction. Longitudinal top flange stresses due to orthotropic action are kept small through the close spacing of transverse floorbeams to support the longitudinal deck stiffeners.

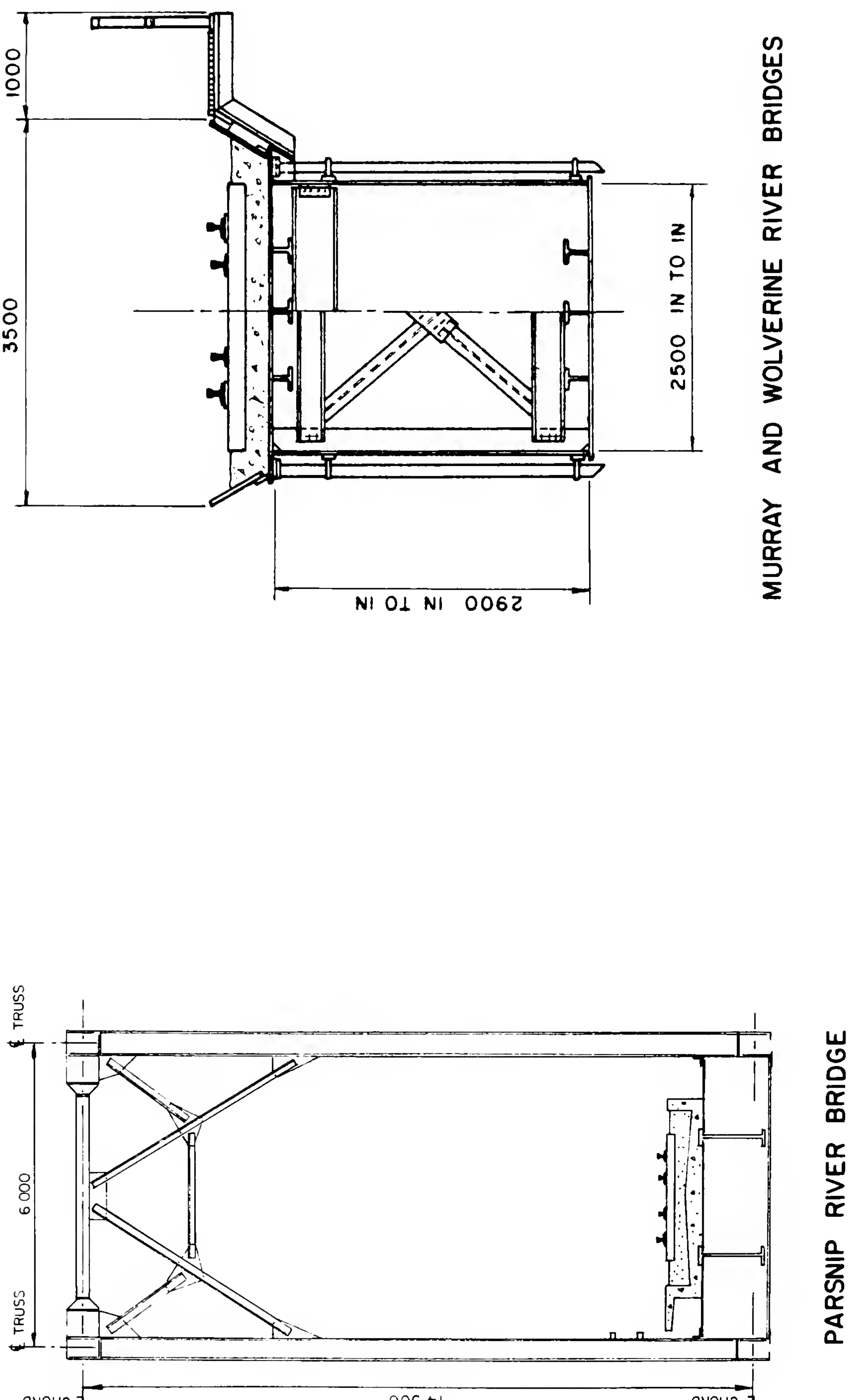

FIG. 4 TYPICAL BRIDGE CROSS-SECTIONS

A.R.E.A. design provisions have been used almost exclusively throughout the bridge design, the only exception being the design of the box girder webs. Since the webs are deep, application of the A.R.E.A. code provisions would result in unusually thick webs. Therefore, the provisions of CSA A1-1966 code "Steel Railway Bridges" have been used to proportion the girder webs. This latter code allows the option of providing a horizontal stiffener to reduce web thickness.

The bottom flange of the box girder is stiffened longitudinally to develop the full allowable compressive stress due to bending in the negative moment region and to reduce transverse bending stresses in the positive moment region. To allow the delta legs to pass up inside the box girder webs, oval cutouts are made in the bottom flange and the bottom flange is reinforced with cover plates to maintain the flange area.

Delta Legs—Murray and Wolverine River Bridges

Each steel delta leg comprises two individual box members connected by large diaphragms at the top and bottom of the leg and a series of intermediate vierendeel struts. The two members act as columns in carrying the vertical and horizontal loads whereas transverse wind loads are carried by vierendeel rigid frame action, the vierendeel frame relying upon its own stiffness to ensure transverse stability.

To make the connection between girder and delta leg, the delta legs pass through oval cutouts in the bottom flange and are bolted directly to the inside of the girder webs. The girder to leg connection transfers large compressive or substantial tensile forces into the two columns of each delta leg. Since the delta legs form a rigid frame with the girder, there are significant restraint forces developed in the delta legs under all types of loading. To relieve these forces, the delta legs are hinged at their connection to the piers, greatly reducing leg moments due to vertical and longitudinal forces.

Under the action of vertical and horizontal loads, the individual box columns of one delta leg carry the load almost equally. Allowance for error in track alignment and the "rolling" term in the live load impact result in the more heavily loaded column carrying 60% of the live load and 54% of the longitudinal forces. In conjunction with the axial loads, moments are induced in the top of the legs as the legs follow the rotation of the girders. However, the delta legs are sufficiently flexible so that moments due to vertical and longitudinal forces consume only 25% of the leg strength. Since the centre of the girder to leg connection is aligned with the neutral axis of the girder, the force transfer from girder web to delta leg is concentric. The steep inclination of the delta legs to the box girder results in only modest axial stresses being developed in the girder through frame action.

Since the box girder is relatively flexible laterally compared to the lateral stiffness of the vierendeel delta legs, almost all wind loading is carried by the delta legs. With no bracing between columns of the delta legs in either the longitudinal or transverse directions, each vierendeel delta leg relies upon its own stiffness for stability. The stiffness of the vierendeel struts and their spacing is such that transverse forces are predominatly resisted by axial loads in the delta leg columns accompanied by small column moments. Under maximum wind conditions, the windward delta leg columns are subject to considerable uplift in the absence of a train on the bridge. Stability of the legs is provided by post-tensioning of the bases of the delta legs to the piers, the post-tensioning force being applied in shear to the webs of the bottom deep vierendeel strut.

In the transfer of load from the box girder web to the outside flange of the delta leg column, substantial eccentric moments would be generated in the column due to eccentricity of load transfer. However, the top vierendeel strut of the delta leg, which also acts as a diaphragm for the box girder, is much stiffer than the delta leg columns and therefore carries most of the eccentric

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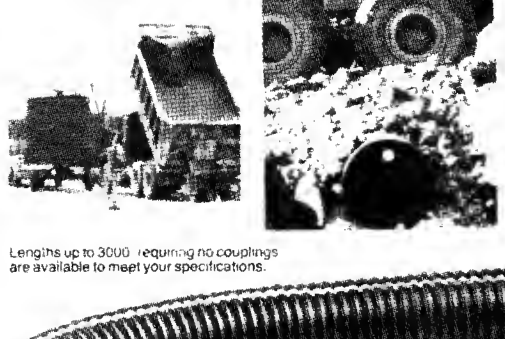

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moment. To facilitate erection the delta legs are spliced just below the underside of the bottom flange so that the delta leg columns and their stiff diaphragm can be shop assembled.

For ease of fabrication, the flanges of the delta leg columns and the webs of the vierendeel struts are one plate, the vierendeel openings being made by cutting out windows in the plate. By suitable proportioning of the vierendeel struts, strut moments are capable of being carried by the webs alone. Thus, diaphragms inside the delta leg columns to develop the full strut flange strength are not required. The strut flanges are only required for stiffness.

Access to the inside of each delta leg column is provided by openings in the top delta leg diaphragm so that entry can be made from inside the box girder. The inside dimensions of the delta leg columns are shown in Figure 5. The end diaphragms of the box girder over the abutment bearings are provided with doors to permit access into the box girder from the bridge abutments.

SUBSTRUCTURES

Parsnip River Bridge Substructure

The Parsnip River Bridge substructure comprises two abutments and a central river pier. All cast-in-place concrete structures are founded on 54 tonne steel H piles. The shaft of the river pier is circular to minimize scour for a skew crossing. The river pier is topped by a rectangular hammerhead cap to support the two truss spans. The river pier was constructed within a cofferdam sealed by a 1750 mm tremie subfooting.

Murray and Wolverine River Bridge Substructures

Because of the inclination of the delta legs in resisting vertical loads and their restraint of longitudinal forces, large horizontal thrusts are transmitted to the substructure. All land piers are founded on spread footings and the vertical compressive forces acting on the land piers are sufficiently large to prevent horizontal sliding through friction generated along the underside of the footings. All abutments and piers are conventionally reinforced with pier caps post-tensioned to resist delta leg uplift. In addition to providing superstructure stability, post-tensioning also increases pier cap durability by suppressing concrete cracking.

Although the end spans are short in comparison to the main spans, the abutment bearings are not subject to net uplift under the action of the superstructure dead load and train loading. However, the extremes of temperature produce net uplift over the abutment bearings. Therefore, holddown bearings are provided at the abutments to resist this extreme loading.

Two alterantive pier footings were designed for the Murray River central pier to enable contractors to bid on either a conventional cofferdam construction or a "bell" form. The possibility of using the bell form arose as the superstructure is comparatively light and "in-river" construction is confined to a small time window in the summer months. By leaving the steel bell form in place, a considerable time saving is made in not having to drive and remove the sheet pile cofferdam. The river pier shaft was made circular to reduce local scour, ice, and stream forces. Both river pier alternatives are founded on vertical piles.

CONSTRUCTION

Parsnip River Bridge Construction

The British Columbia Railway Co. awarded the 5.2 million dollar contract for the construction of the Parsnip River Bridge to Peter Kiewit and Sons Co. Ltd. of Richmond, B.C. in August 1981. The contractor obtained special permission to construct a gravel berm past the

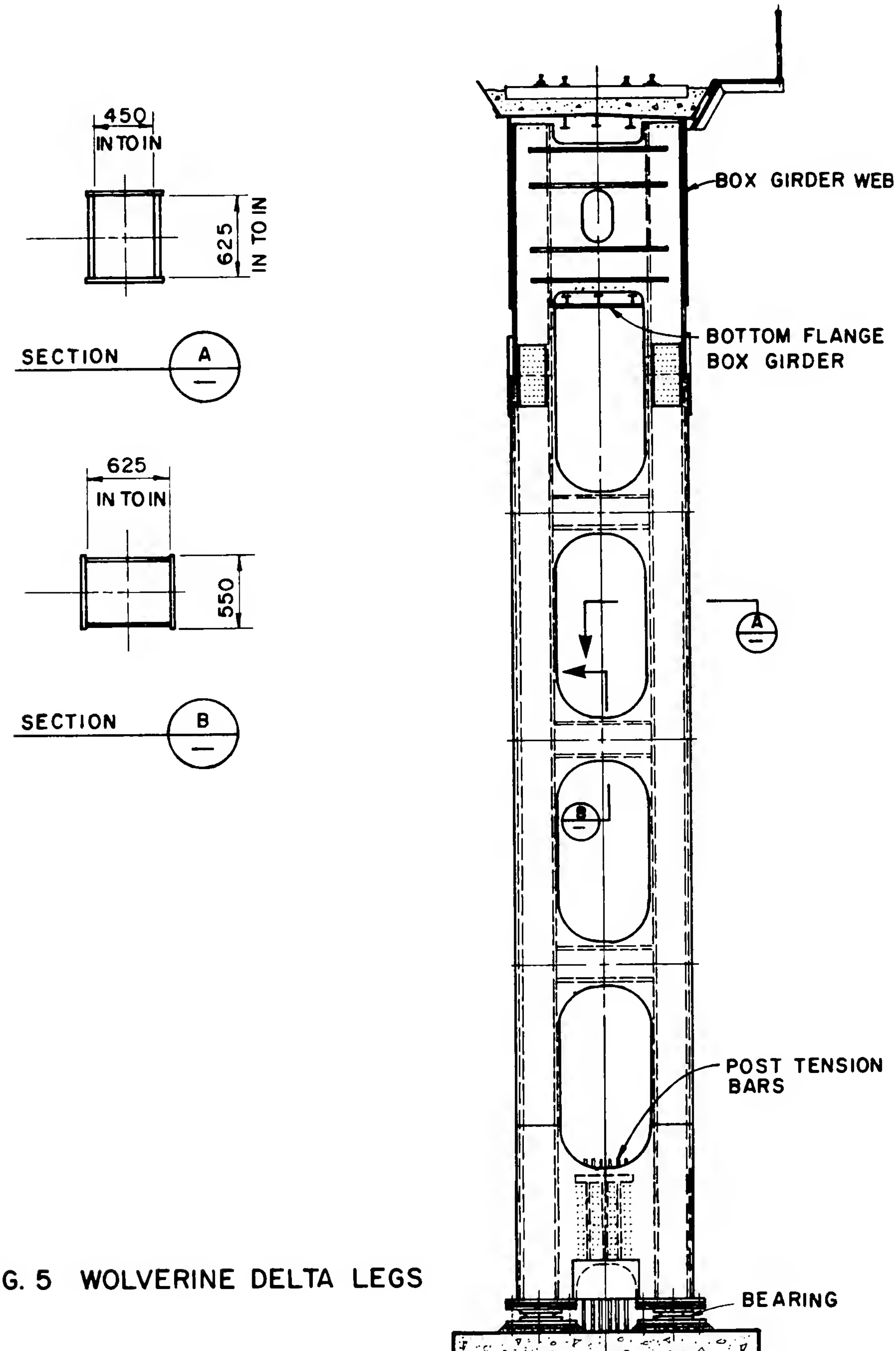


FIG. 5 WOLVERINE DELTA LEGS

midpoint of the river on the upstream side of the river pier. The berm combined with a geotextile silt fence draped from a log boom formed an environmentally isolated basin in which a glory hole was excavated and a cofferdam constructed for the 1.4 million dollar river pier. A 2.5 metre thick tremie concrete plug sealed the bottom of the nominally 13.5 metre square sheet pile cofferdam. A commercial retarder was used to prevent cold joints in the 460 cubic metre pier footing as the portable concrete batch plant capacity was only 19 cubic metres per hour. Construction of the east abutment wing walls was deferred and the east approach fill grade was kept 1.5 metres low to accommodate the superstructure erection.

The 2.4 million dollar structural steel subcontract was executed by Brittain Steel Ltd. of New Westminster, B. C. The 14 week long erection scheme started in April of 1982 avoided the flood-swollen Parsnip River by assembling the steelwork on timber cribbing on the east approach fill and then launching to the west abutment. The two spans were bolted together at the bottom chords and specially fabricated tie beams linked the top chords together to create a temporarily continuous structure. Pairs of 150 mm wide by one metre long Teflon pads formed sliding shoes at each of the panel point locations. 19 metre long rocker girders at the east abutment and river pier together with shorter tail girders founded on the east approach fill formed the bottom sliding surface. The tail-end-ballasted steelwork was launched so that the 84 metre cantilevered leading span first engaged the river pier rocker girders and then the west abutment. The 720 tonne superstructure was hydraulically raised to remove the rocker girders before carefull lowering on to the bearings.

The formwork for the cast-in-place ballast trough was completely prefabricated to meet the scheduled August 1982 completion date of the project.

Murray River Bridge Substructure Construction

The 1.2 million dollar contract for the Murray River Bridge substructure was awarded to Goodbrand Construction Ltd. of Aldergrove, B. C. in July 1982. The contract value was considerably less than the engineer's estimate, a strong reflection on the prevailing economic times. The bell footing river pier alternative had no significant cost advantage over the conventional cofferdam construction and hence the latter was chosen.

A timber pile trestle accessed a 9.5 metre square sheet pile cofferdam at the river pier site. The 5.5 metre deep pier excavation in very dense glacial till took over two weeks to complete. Following pile driving, a 2 metre thick tremie seal was cast by direct pumping of the concrete through the vertically submerged slick line of the pump truck.

A segmental prefabricated plywood form strengthened with tensionable steel bands was used to construct the truncated cone atop the pier column.

The contractor chose to widen and ditch the land pier excavations in order to control the water infiltration and hence dewatering costs were minimized for these relatively shallow footings. Large metric sized Efco panels proved to be most economical in forming the land pier footings and pedestals. The pedestals were cast in two lifts to enable positioning of the Dywidag post-tensioning rods.

The abutment footings and bridge seats were cast in 1982 and the wing walls were completed in the summer of 1983 following the superstructure erection. Approach fills were by others.

Wolverine River Bridge Substructure Construction

Almost one quarter of the $830,000 contract awarded to Goodbrand Construction Ltd. for the Wolverine River Bridge substructure was for dewatering the two land piers. The permeable gravels overlying the poorly defined bedrock obviously were of concern to the contractor.

Dewatering of the land based pier subfootings was accomplished with a sand bag perimeter dyke on the high sides of the sloping competent bedrock foundation. The piers were constructed in a manner similar to that of the Murray River Bridge. Following completion of the approach fills by others, the abutment footings and bridge seats were cast in the late fall of 1982, whereas the wing walls were completed the following year after superstructure erection.

Murray and Wolverine River Bridge Superstructure Erection

Canron Inc. Western Bridge Division of Vancouver, B. C. was the successful tenderer for the 3.5 million dollar superstructure contract for the Murray and Wolverine River Bridges awarded by BCR in August of 1982. The 1000 tonnes of steel were fabricated in three months. Each of the box girders was fabricated in nine sections, whereas the delta legs were singly constructed. A five month pre-construction lead time was vital to the contractor who chose a launching method of erection. Permanent changes to the structure required to accommodate the launching stresses needed to be identified prior to ordering the plate steel. Arrangements had also to be made to leave the launching bed grade 3 metres low and to omit the abutment wing walls at the launching side of each bridge.

The January 1983 erection started with the installation of temporary steel bents on the five piers of the two bridges. The bents were stabilized by guying to the adjoining piers and abutments. The delta legs were then erected and supported from the temporary bents.

Pairs of 3.6 metre long cambered rocker beams on which the superstructure would slide were mounted on the launching side abutment and each of the temporary bents. The bent mounted rocker beams were moved transversely outwards to permit the deflected box girder to pass between the 30 mm wide sliding surfaces. The very narrow sliding surfaces were necessary because of the delta leg stubs projecting from the underside of the box girder.

The box girder sections were assembled on sleds resting on a steel clad timber rail and timber sleeper skidway. Extremely tight tolerances were demanded of the fabrication and assembly of the box girder so that the bottom flange edge would bear evenly on the rocker beam sliding surfaces during launching.

The staged launching consisted of pushing the box girder to the first pier bent, jacking the girder nose upwards over the first pier bent to restore erection geometry and to set the box girder on the rocker beams, and then launching to the next pier and the sequences repeated. Following completion of the launching the delta legs were jacked up to mate with the box girder.

Particular attention was required in setting the final geometry of the superstructure prior to grouting the bearings and post-tensioning in order to minimize locked-in stresses. The superstructures were substantially completed in June of 1983.

CONCLUSION

The design of major railway bridges has changed very little in British Columbia in the past decade. This paper describes contrasting structural systems for different bridge sites, illustrating conventional and unusual innovative approaches adopted in crossing the three larger rivers on the British Columbia Railway's Tumbler Ridge Branch Line.

The low-level Parsnip River Bridge comprises conventional through steel truss spans with a cast-in-place ballasted concrete deck. The medium-level Murray and Wolverine River Bridges utilize a ballasted steel box girder supported by a pair of steel delta legs over each pier. Stability of the delta legs is provided by post-tensioning of the leg bases to the pier caps.

Report on May 1983 Delegation of A.R.E.A. Members in China

L. T. Cerny*

FIG. 1 Delegation walks track in vicinity of Great Wall of China north of Beijing.

From May 9th to May 28th 1983, an AREA delegation comprised of 32 members and 12 wives, led by AREA President Terrill and Executive Director Cerny, participated in a technical and cultural exchange with railway engineering officials from the Peoples' Republic of China.

Invited by Chinese officials to see, first hand, the country's vast railway network and its current engineering practices, the delegation travelled about 2000 miles by rail, beginning with

FIG. 2 Beijing-Tianjin passenger train a few miles northwest of Tianjin.

*Executive Director A R E A and Engineering Division, Association of American Railroads

FIG. 3 Throat of main Beijing station which is stub-ended and completely dieselized.

the capital city of Beijing at the coast, and continuing southwest deep into the mountainous interior via the cities of Zhengzhou, Xian, Chengdu and Kunming.

Because of the many similarities which exist between the North American and Chinese railway networks—the same track gauge, identical coupling and basic air brake systems, and an extensive use of wood ties, with cut spikes and tie plates—a special professional bond exists

FIG. 4 Activity at station in Xian, China.

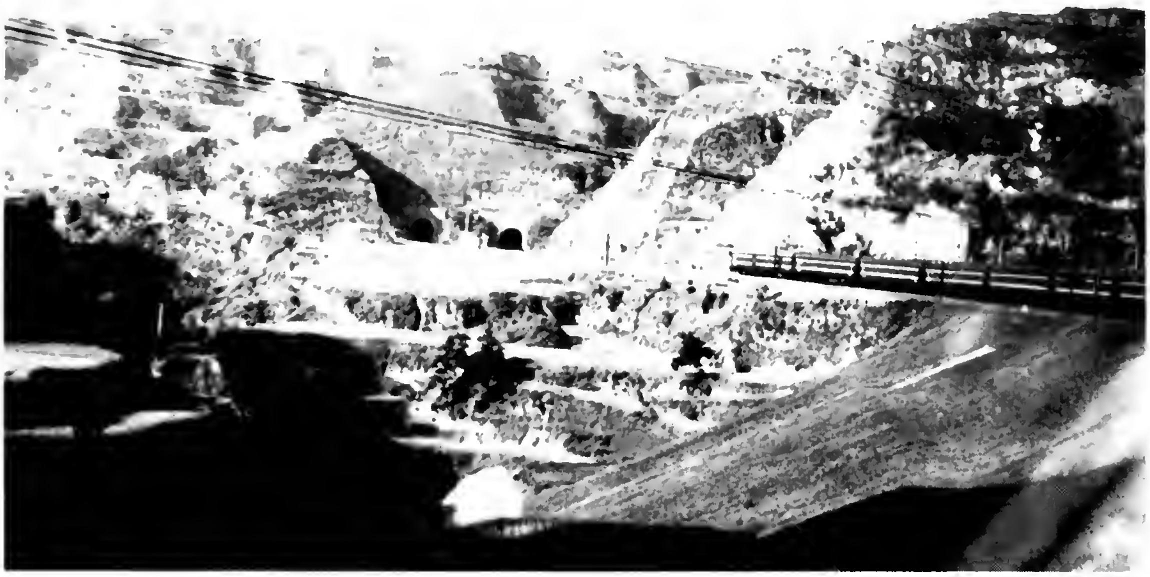


FIG. 5 View from window of sleeping car on eastern portion of Zhengzhov-Xian line.

between the railroaders of both countries. It did not take long before the AREA track experts felt right at home looking down at the familiar tie, spike and rail configuration.

There are, however, differences. China does have a much higher percentage of concrete ties than the U.S. In addition, maximum allowable axle loads limit a four-axle freight car to 185,000 pounds gross weight. Rail weights vary from 86 to 120 pounds per yard.

FIG. 6 View from window of sleeping car on Zhengzhov-Xian line near Xian.

FIG. 7 **Train with delegation of A.R.E.A. members heads up 3 percent grade between Baoji and Chengdu. The bridge above the electric locomotive is on the same line as the train.**

At a seminar held near the mountain resort of Emei during the last few days of the exchange a panel comprised of four AREA members who are or were chief engineers in the U.S., concluded in a joint opinion that they had not seen any track in China incapable of handling 220,000-pound (nominal 70-ton capacity) cars of U.S. design.

FIG. 8 **Chinese track inspector in hump yard near Tianjin. Delegation members are in background.**

FIG. 9 400 ft. high statue of Buddah near Leshan, China.

Optimum use of China's labor resources often involves large manual gangs. In comparing track quality, the Chinese track was deemed equivalent to good U.S. mainline track.

Maximum utilization of resources has prompted the Chinese to continue the manufacture of steam locomotives along with diesels and electrics. Almost 80 percent of all locomotives in China are steam-powered, though some significant key line segments are almost totally dieselized or electrified.

The main Beijing passenger station, for instance, sees only diesel-powered trains. (See Figure 3) In Xian, all 162 daily trains, as well as the switching, are steam-powered. (See Figure 4 and pages 1-3 and front cover of Bulletin 694) For the most part, the delegation travelled behind a diesel locomotive, (See Figure 5) though for about 300 miles, the group rode on an electrified line, and on a 200-mile stretch, experienced the pulling power of steam. (See Figure 6)

Many of China's outstanding accomplishments in railway civil engineering have come in the area of tunnel and mountain construction, such as the Baoji-Chengdu line. (See Figure 7)

The Chengdu-Kunming line, completed in 1970, includes 200 miles of tunnels and 60 miles of bridges within its 700-mile stretch. Despite side hill locations on steep rock slopes and in twisting canyons, the line maintains a maximum curvature of 6 degrees.

Chinese tunnel use includes reversing direction inside a mountain to avoid switchbacks and as a method to gain additional distance in order to maintain the maximum design grade.

Aside from having the opportunity to examine China's railway system and engineering practices, including marshalling yards (See Figure 8) the AREA delegation was also able to enjoy the country's noted cuisine and arts. Sights like the Great Wall, the Terra Cotta Army at Xian, the 400-foot Buddha near Leshan, (Figure 9) all enhanced the group's appreciation and awe of Chinese history and culture.

MARTA's Rapid Transit Bridges — the first segmental concrete railway project in the U.S.

J. M. Muller*

The first precast segmental concrete railway structures in the United States have recently been completed in Atlanta, Georgia, for the Metropolitan Atlanta Rapid Transit Authority (MARTA). They represent significant design and construction contributions which will promote the development of rapid transportation systems in the U.S.

The contractor, J. Rich Steers, Inc. of New York, was the low bidder on two projects for MARTA (two elevated structures with a combined total of over 7,000 linear feet). Contract documents allowed the contractor to submit a redesign in segmental concrete. Steers hired Figg and Muller Engineers, Inc. (headquarters in Tallahassee, Florida), which specializes in the design of concrete segmental bridges, to perform this redesign.

Figg and Muller Engineers' redesign was based on precast concrete segmental construction erected by the span-by-span method with external post-tensioning tendons (tendons are located within the box girder void, but external to the concrete).

A new assembly truss concept was developed for this project which has proved to be a real "breakthrough" for the construction of 70' to 140' spans in highly congested urban areas. The new assembly truss resulted from an innovative modification of the original truss developed for the Long Key and Seven Mile Bridges in the Florida Keys. Instead of a single truss beneath the superstructure supporting the segments at the bottom slab of the box, the MARTA trusses are triangular and are located on each side of the box section, supporting the box girder under the wings. Through utilization of the triangular trusses for span-by-span erection, it was possible for the contractor to complete one span per day. This is a decided advantage over cast-in-place on falsework, in which it would take at least five days to complete one span.

During the span-by-span erection process, the segments were placed on the twin triangular trusses by crane. (It took approximately one hour and 15 minutes to place all the segments for an 80' span.) The wings of the segments rested on movable supports that were on the top flange of the triangular trusses.

Splitting the trusses makes them lighter and more maneuverable, which facilitates faster movement. (It took approximately 45 minutes to an hour to move both trusses.)

CS 360 Construction

The first elevated structure substantially completed was CS 360. At 5,230' in length (158,200 square feet), it is in the MARTA South Line which connects downtown Atlanta with Hartsfield International Airport, running parallel along Highway 29 for much of the way. At 30.25' wide, CS 360 is designed to carry two trackways. The box girder superstructure segments are 10' long, 7' deep and weigh approximately 30 tons. CS 360 consists of simple spans, ranging from 70' - 100' in length.

Erection of the superstructure segments began in November, 1982, and the construction pace exceeded all expectations. Up to four 80' spans were completed per week, (300'+ per week) requiring only sound barriers and track installation to be ready for service.

* Chairman of the Board and Technical Director, Figg and Muller Engineers, Inc

Innovations in Casting

The casting yard for the box girder segments was located two blocks from the CS 360 project, in an area which will become a future parking lot for a passenger station. Three long-line casting beds were set up -- the first in the U.S. for full span -- which turned out up to 20 segments per week. Match-casting of joints was achieved by moving side forms and a core form down the bed as the segments were cast.

A major advantage of this system is the removal from the critical path of a high concrete strength requirement before handling segments. The segments can remain on the beds while the side forms are moved to the next segment. Also, the movable side forms were equipped with the steel frame on top developed by J. Rich Steers which allowed the top slab to be transversely pretensioned -- another innovative feature.

To accomplish curvature, the bottom slab chorded the span length, while the wings were adjusted to obtain the curvature necessary for the tracks. Thus, the bottom soffit became straight, which allowed placement on the long-line bed, and the wings were easily cast to the correct dimensions by adjusting the metal forms.

CN 480 Construction

The CN 480 structure is part of MARTA's North Line extension which terminates in Doraville. (CN 480 is 1,900' long; 57,500 square feet). Since CN 480 is similar to the CS 360 structure, Steers was able to use the same casting and erection equipment. The major difference between the two structures is that CS 360 consists of all simple spans, but CN 480 includes a four-span continuous unit. (Span lengths vary from 75' to 143".)

To accommodate the variety of span lengths, the truss was adapted with the addition of 40', 8'-10" or 5' sections; a C-hook was added to facilitate the movement of the truss.

The segment dimensions for the two projects are identical, so segments were cast at the CS 360 site and transported across town. Portability of the precast segments is an advantage, and through the use of converted six-ton trucks (a very maneuverable method of transportation) and low-boy trailers, there were no restrictions imposed, such as night transport.

Continuity in the four-span unit was accomplished by modifying the post-tensioning patterns for the pier segments, where the tendons are anchored. The outside concrete dimensions were not revised, allowing the use of the same side forms for both sections.

Ramifications for Future Rapid Transit Projects

The following design and construction techniques are incorporated in the CS 360 and CN 480 structures; saving money and time while still providing the best bridge possible:

- multiple shear keys in the box webs;
- span-by-span erection;

- match-cast segments with no epoxy in the joints;
- transverse pretensioning of the top slab;
- external post-tensioning with tendons within the box girder void, but external to the concrete.

The MARTA Project demonstrates the ability of the segmental technique to economically solve bridge construction problems in heavily congested areas, which is important to other proposed mass transit projects in the U.S. An example of the progress Figg and Muller Engineers made in span-by-span construction techniques is that the assembly truss on MARTA cost only about one-quarter of the truss used on the Long Key Bridge (designed in 1976).

Segmental concrete bridges offer strong advantages in cost-effectiveness, low maintenance requirements, and fast erection capability. They can be built in sensitive areas with a minimum of disturbance to the environment, and in highly congested areas with little disruption to the flow of urban traffic. In addition, the aesthetics achieved on the MARTA bridges -- the pleasing appearance of the underside of these concrete boxes with their cantilever wings -- make the bridges a welcome part of Atlanta's transportation network.

FACT SHEET

MARTA Rapid Transit Bridges
Atlanta, Georgia

The First Precast Segmental Concrete Railway Structures in the U.S.
MARTA CS-360 and MARTA CN-480

LOCATION:	Atlanta, Georgia
CLIENT:	J. Rich Steers, Inc.
OWNER:	Metropolitan Atlanta Rapid Transit Authority (MARTA)
ENGINEERS:	Parsons, Brinckerhoff/Tudor for MARTA *Redesign by Figg and Muller Engineers, Inc.* for J. Rich Steers, Inc.
TOTAL LENGTH:	CS-360—5,230′ CN-480—1,900′
SPAN LENGTH:	CS-360—varies from 70′ to 100′CN-480—varies from 75′ to 143′
WIDTH:	30.25′ overall. Two trackways.
SUPERSTRUCTURE:	Precast segmental box girder. Constant depth of 7′. Superstructure boxes were cast using long-line method. Contractor used 3 casting machines and cast as many as 20 segments per week.
SUBSTRUCTURE:	Reinforced concrete piers on piling (by others)
QUANTITY:	CS-360—Total deck area—158,200 s.f. Total precast segments—672
	CN-480—Total deck area—57,500 s.f. Total precast segments—249
SUPERSTRUCTURE ERECTION:	The introduction of a new triangular erection truss concept allowed the contractor to complete as many as four spans per week (up to 320′ of superstructure completed in one week).
	In addition to speed of construction, the other outstanding benefit of this new method is that there is no need to interrupt the traffic moving beneath the construction. This is a cost-effective solution for congested areas, and one which will cause only a minimum disruption for all types of traffic.

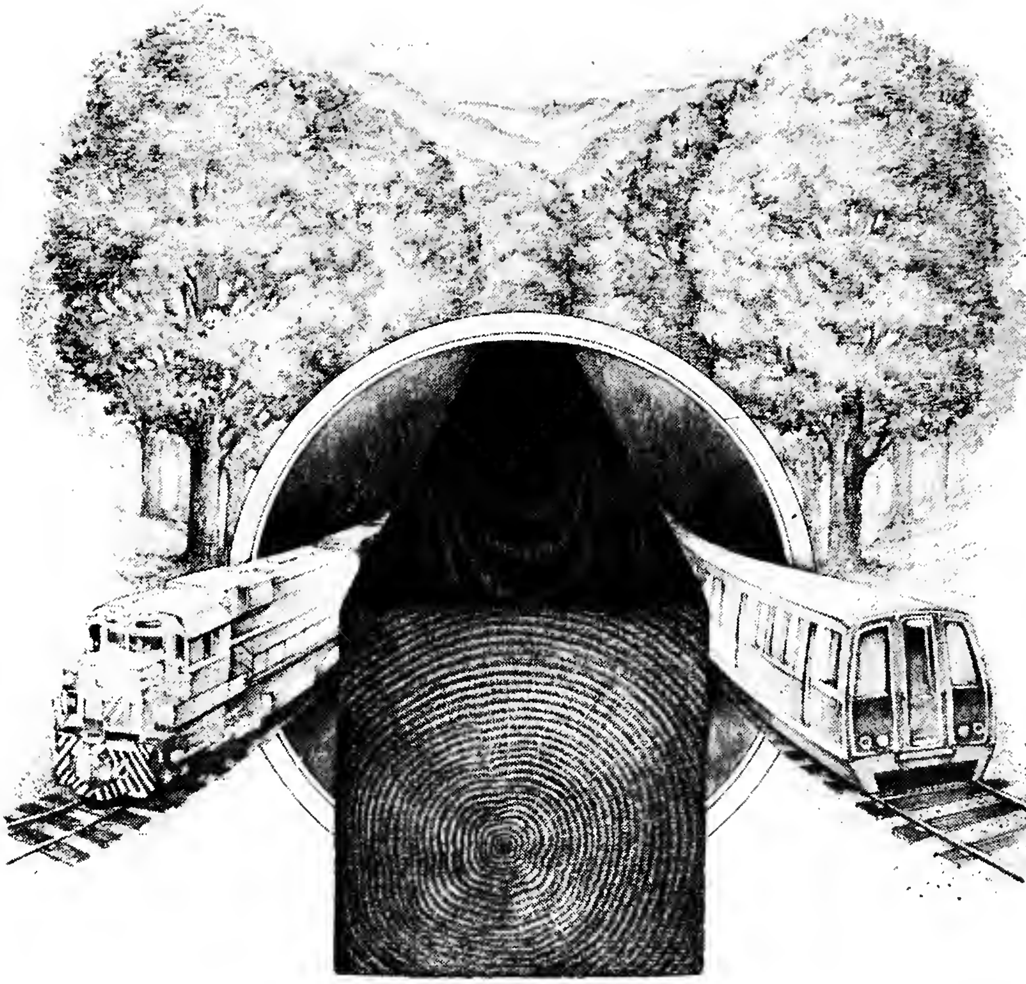
MARTA 360 and 480 RAPID TRANSIT PROJECT (360 Project shown)
Atlanta, Georgia

J. Rich Steers, Inc. hired Figg and Muller Engineers, Inc. to re-design the superstructure for 7,000 l.f. of elevated railway in precast concrete segmental. Box girders are 30.25′ wide, 7′ deep, providing two trackways. Shown: Segment being placed. Span-by-span erection through utilization of two triangular erection trusses has enabled the contractor to place up to four spans of up to 100′ in length per week. This new "truss breakthrough" is important to 70% of all the bridges built in the U.S. because this construction concept allows the construction of 70′ to 140′ spans in highly congested urban areas—without interruption of traffic.

J. R. Steers' precast yard for the box girder segments is located two blocks from the CS 360 project. Long-line casting beds are first in U.S. for full span. Up to 20 segments cast per week on 3 casting beds.

CS-360 MARTA RAPID TRANSIT PROJECT
Atlanta, Georgia

No special equipment was required to transport the segments from the casting yard. They were delivered to the site by truck and lifted onto the triangular trusses by crane.

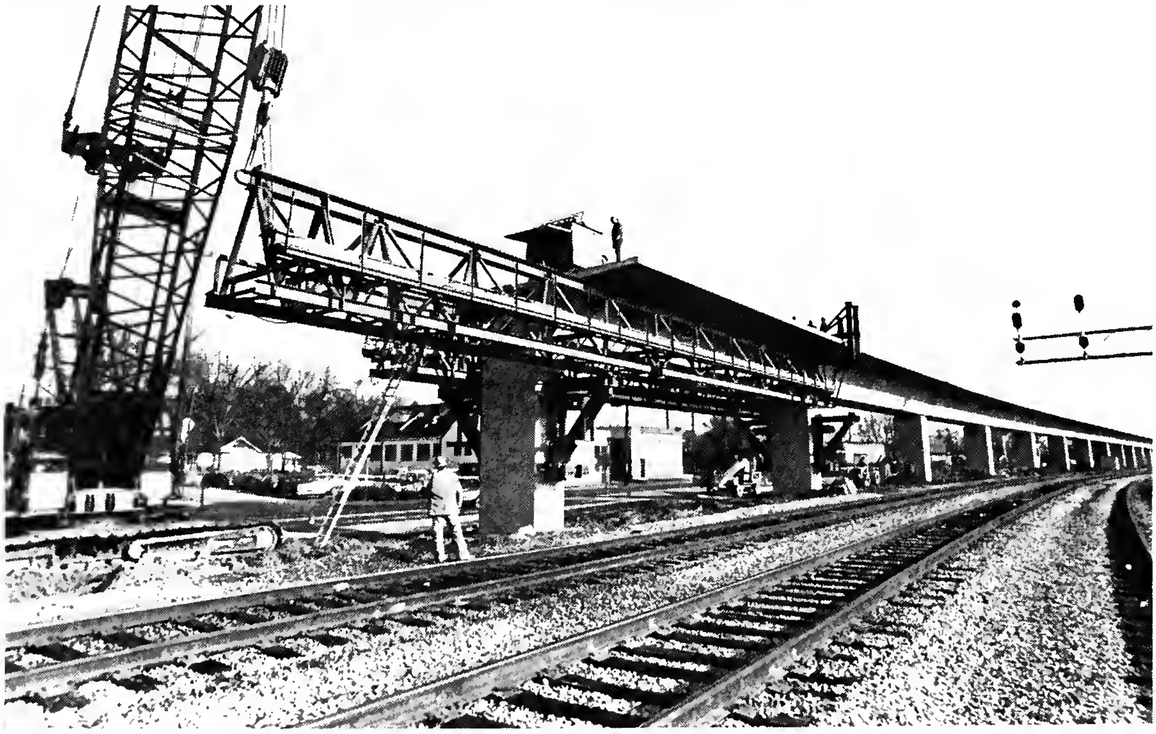


MARTA 360 RAPID TRANSIT PROJECT
Atlanta, Georgia

Segment being placed on twin triangular trusses. It took J. R. Steers staff approximately 1 hour and 15 minutes to place all segments for a 80′ span. Up to four spans were completed per week.

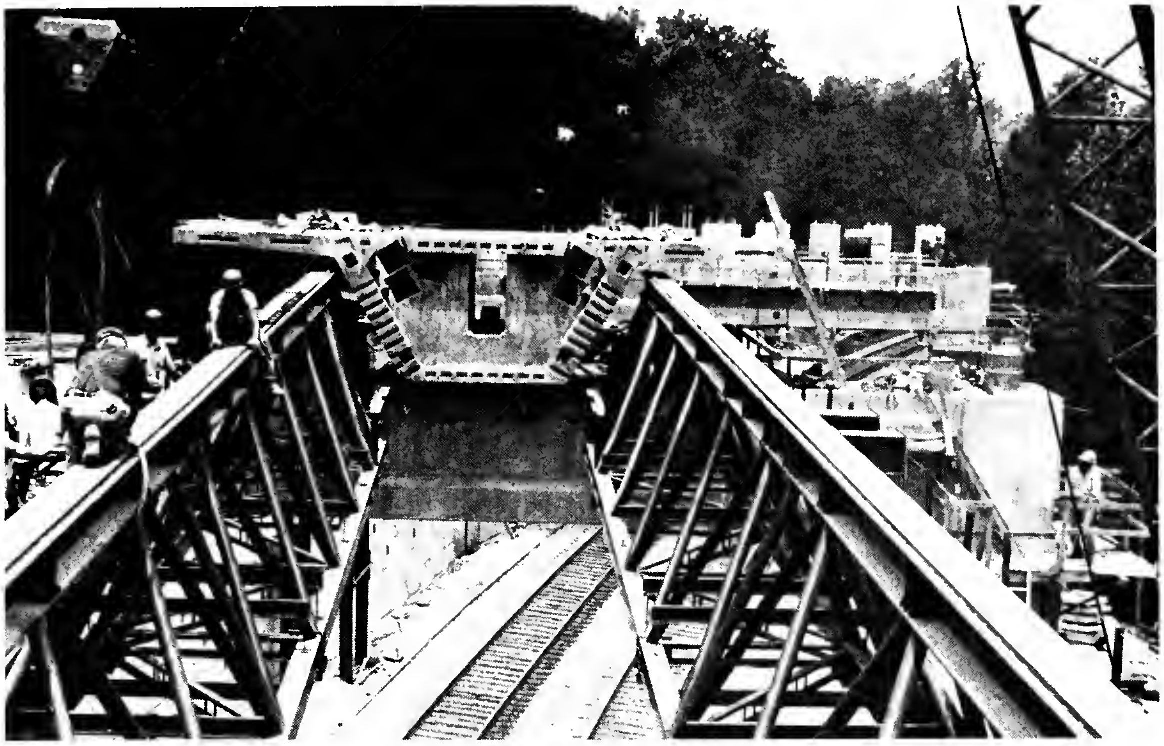
MARTA CS-360 RAPID TRANSIT PROJECT
Atlanta, Georgia

The wings of the segment rest on movable supports that are on the top flange of the triangular trusses. Segments are winched into their proper place in the span.



CS-360 MARTA RAPID TRANSIT PROJECT
Atlanta, Georgia

One of the twin trusses being moved. It takes approximately 45 minutes to an hour to move both trusses.



CN-480 MARTA RAPID TRANSIT PROJECT
Atlanta, Georgia

Approximately 370′ of CN-480 had to be built over an operational Southern Railway Line.

CN-480 MARTA RAPID TRANSIT PROJECT
Atlanta, Georgia

Span has just been completed and the trusses are ready to be moved for construction of the next span, as the superstructure curves gracefully overhead.

Marne la Vallee Viaduct, France

First precast segmental railroad bridge in France, designed by Jean Muller. 193,700 s.f. Ninety-five spans ranging from 70′ to 125′ long and 36′ to 52.5′ wide. Segments were built in casting cells and placed with a launching girder. Built in 1977, it is an extension of the Paris mass transit system. Project included a bridge over the Marne River and a long urban viaduct carrying two parallel railway tracks.

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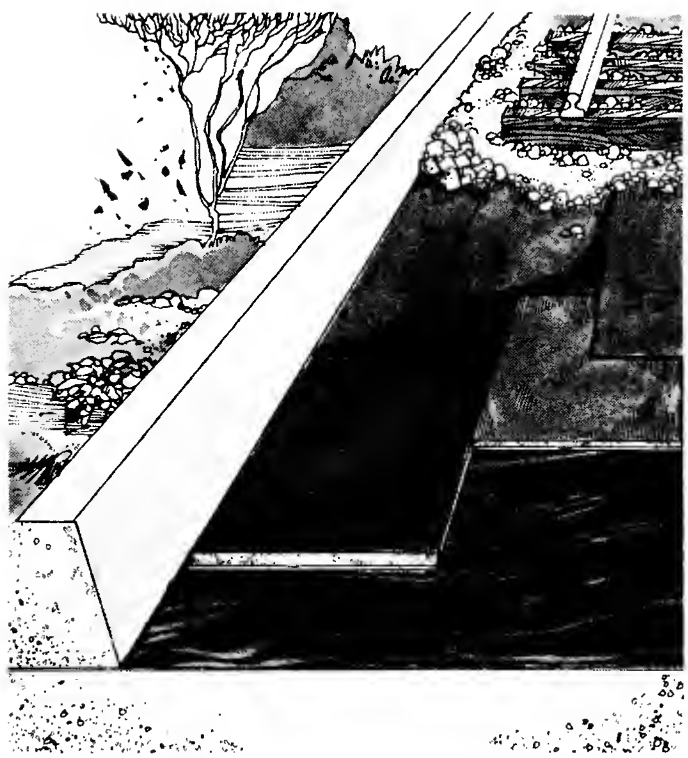

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COMMITTEE 3—TIES AND WOOD PRESERVATION

ADVANCE REPORT ON ASSIGNMENT 5
SERVICE RECORDS

F. J. Kotroba (Chairman, Sub-Committee); H. C. Archdeacon; C. A. Burdell; L. A. Buell; L. C. Collister; M. J. Crespo; E. M. Cummings; H. C. Edscorn; G. C. Farkas; E. H. Hilderbrand; J. E. Hinson; L. G. Hope; D. B. Mabry; H. F. Ninstill; H. E. Richardson; G. D. Summers; G. H. Way; J. G. White

Statistics providing information on cross tie renewals for 1982 as compiled by the Economics and Finance Department, Association of American Railroads are presented in the attached table.

By geographical districts, the Eastern Roads inserted in replacement 75 ties per mile; the Southern Roads 74 ties per mile and the Western Roads 68 ties per mile. Average for the United States was 71 ties per mile.

"Indicated" wood tie life determined by dividing the total number of ties in track ('67 figures) by the number of new ties inserted in 1982 is as follows:

Eastern Roads	40 years
Southern Roads	42 years
Western Roads	45 years
All US Class I Roads	43 years

Apparently, some maintenance was deferred in 1982. New tie insertions remained fairly stable in years 1978 through 1981, but dropped significantly in 1982. The 1982 drop in new tie insertions represented a 20% decrease of the 1981 figure of 89. Undoubtedly, the business recession and subsequent decrease in freight revenues were partially responsible for this decrease. However, the better-maintained roads could easily afford the 1982 decrease without serious effects on track stability. 1983 and '84 should see new tie insertions returning to the more normal figure of 88-90 per mile.

Year	Total New Tie Renewals	Total Miles Occupied By• Crossties	Renewals Per Mile
1978	25,033,738 (1)	284,244	88
1979	24,262,740 (2)	274,279	89
1980	23,691,171 (3)	270,510	88
1981	23,627,532 (4)	267,624	89
1982	18,522,737 (5)	262,890	71

(1) Does not include 41,985 concrete ties (Florida East Coast) and 769,656 second-hand ties.
(2) Does not include 79,680 concrete ties (Florida East Coast) 297,820 concrete ties (Amtrak-Northeast Corridor) and 681,688 secondhand ties.
(3) Does not include 322,295 concrete ties and 1,005,096 second-hand ties.
(4) Does not include 313,285 concrete ties and 1,299,276 second-hand ties.
(5) Does not include 258,332 concrete ties and 925,595 second-hand ties.

Crosstie Statistics (excluding switch & bridge) for Class I Railroads in the United States

Calendar Year Ended December 31, 1982

	Wooden cross ties *laid in replacement*		Track maintained by *reporting railroad*			New cross tie *replacement averages*	
District and Road	New Ties	Second hand ties	Miles occupied by crossties	Total cross ties	Cross ties per mile (1967)	Percent renewal to all ties	Number laid per mile
EASTERN DISTRICT							
Baltimore & Ohio	964,946	8,582	9,041	26,218,900	2,900	3.68%	107
Bessemer & Lake Erie	6,131	—	432	1,296,000	3,000	.47	14
Boston & Maine	42,117	3,127	1,569	4,628,550	2,950	.91	27
Chesapeake & Ohio	937,596	5,480	7,797	23,391,000	3,000	4.01	120
Conrail	1,807,212	186,356	31,299	93,740,505	2,995	1.93	58
Delaware & Hudson	80,035	2,291	1,241	3,847,100	3,100	2.08	64
Detroit, Toledo & Ironton	41,421	—	509	1,465,920	2,880	2.83	81
Elgin, Joliet & Eastern	12,960	—	884	2,703,272	3,058	0.48	15
Grand Trunk Western	131,911	—	2,065	6,508,880	3,152	2.03	64
Long Island	52,255	276	711	2,011,419	2,829	2.60	73
Norfolk & Western	1,060,397	—	13,145	40,894,095	3,111	2.59	81
Pittsburgh & Lake Erie	37,764	—	668	2,043,412	3,059	1.85	57
Western Maryland	82,071	2,220	941	2,730,782	2,902	3.01	87
Total Eastern District	5,256,816	208,332	70,302	211,479,835	3,008	2.49	75
SOUTHERN DISTRICT							
Clinchfield	136,663	—	483	1,564,920	3,240	8.73	283
Florida East Coast	11,628	—	859	2,589,885	3,015	2.50	75
Illinois Central Gulf	1,271,401	—	11,956	37,936,388	3,173	3.35	106
Louisville & Nashville	731,335	—	9,786	28,222,824	2,884	2.59	75
Seaboard Coast Line	846,394	—	13,641	42,464,433	3,113	1.99	62
Southern System	678,507	66,430	13,450	41,856,400	3,112	1.62	50
Total Southern District	3,675,948	66,430	50,175	154,634,850	3,082	2.47	74
WESTERN DISTRICT							
Atchison, Topeka & Santa Fe	876,222	11,087	19,601	62,585,993	3,193	1.40	45
Burlington Northern	3,722,169	4,056	40,170	122,558,670	3,051	3.04	93
Chicago & North Western	403,182	387,546	11,059	32,966,879	2,981	1.22	36
Chicago, Milw., St. P. & Pac.	225,462	101,690	4,521	13,712,193	3,033	1.64	50
Denver & Rio Grande Western	258,654	446	2,756	8,507,772	3,087	3.04	94
Duluth, Missabe & Iron Range	1,356	909	718	2,138,922	2,979	0.06	2
Kansas City Southern	399,702	—	2,393	7,655,207	3,199	5.69	182
Missouri-Kansas-Texas	252,188	—	2,456	7,869,024	3,204	3.20	103
Missouri Pacific	1,249,929	—	15,110	46,508,580	3,078	2.69	83
St. Louis Southwestern	151,493	—	3,184	9,752,592	3,063	1.55	48
Soo Line	180,137	—	5,421	16,306,368	3,008	1.10	33
Southern Pacific	1,128,166	—	16,503	48,683,850	2,950	2.34	69
Union Pacific	550,585	57,122	14,165	40,044,455	2,827	1.37	39
Western Pacific	127,153	1,196	1,878	5,605,830	2,985	2.27	68
Total Western District	9,526,398	564,052	139,935	424,896,335	3,036	2.25	68
Total United States	18,459,160	838,814	260,412	791,011,020	3,038	2.35	71
Amtrak	63,577	86,781	2,478	7,528,164	3,038	2.96	90

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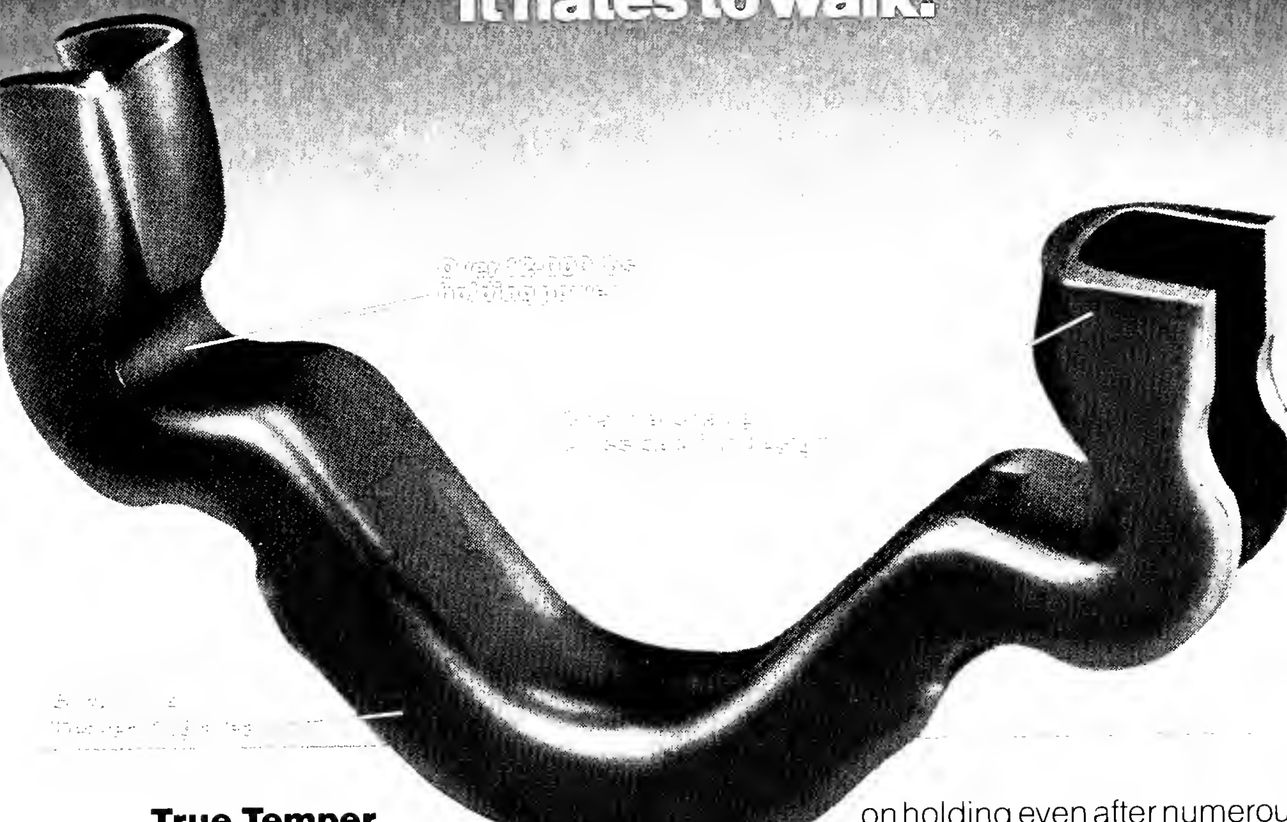

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COMMITTEE 14—YARDS AND TERMINALS

Report on Assignment 5

Design of TOFC/COFC Facilities

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I. INTRODUCTION

Trailer on flat car/container on flat car (TOFC/COFC) terminals are specialized freight terminals which receive, manage and dispatch empty or loaded highway semi-trailers or semi-trailer chassis-mounted cargo containers in intermodal service. TOFC/COFC terminals integrate rail, highway and waterway transportation modes, thus giving the shipper access to the benefits of the rail mode while retaining the advantages of the highway or waterway modes.

A. General

Factors influencing the facility location and design are anticipated traffic volume, its origin and destination within the service area, the local rail-highway network configuration and the location of existing rail facilities required to service the new terminal. Interface with highway and water transportation is quite important. The location studies must consider the equipment type, the traffic volume, railroad operations, highway traffic patterns and a commitment from railroad management concerning the general area where a TOFC/COFC terminal is desired. The justification for installing technologically advanced equipment will depend on the size and expected activity in the proposed facility.

Marine terminals usually handle large volumes of containers between the marine and rail modes, while inland terminals usually handle smaller volumes of containers between the highway and rail modes. For further information and typical design drawings on container terminals, refer to the report of Committee 14, Assignment 5 entitled "Terminal Facilities for Container Handling", in *Bulletin No. 635*, Proceedings Volume 73, Nov-Dec 1971.

1. Operational Concept

The design of a TOFC/COFC terminal will be governed principally by the selected operational concept determined by considering the information developed during the feasibility studies, i.e., the volume of traffic, the land available and the special problems that occur when a facility handles both trailers and containers because of their different equipment and storage requirements.

Several factors determine the actual terminal operating procedures. The number of trailers and containers to be loaded and unloaded has an important effect on procedures and the service offered by the facility plays a major role in the operation. Waterfront terminal serving a port handling container ships usually requires more storage/parking area than an inland terminal. Operations commonly handling perishable loads have different needs since electrical outlets and additional diesel-fuel supplies are required to keep the refrigerated units running.

For efficient terminal operation, standards for the specific facility and parking procedures should be developed to permit efficient truck-trailer movement from the terminal gate to a designated parking location. This procedure can significantly improve the terminal operation. A computer program to help locate and retrieve trailers and containers will permit more efficient location assignments for incoming and outgoing trailers. Most large terminals have access to digital computers programmed for this purpose.

Centralized management information systems will facilitiate trailer handling, spotting, pre-blocking and all associated paper flow.

B. Site Selection Planning

Many of today's TOFC/COFC terminals were simply located on a site available to the railroad with a layout made to conform to the available space, thus causing cost and service penalties for internal storage and retrieval, loading and unloading, improper circulation, switching, and security. This approach should be avoided if at all possible.

Various alternative layouts and order-of-magnitude costs must be developed for each site with their advantages and disadvantages weighed objectively and subjectively to determine the most cost-effective site. The end product of this planning will be a report to railroad management discussing each alternative site and recommending the most attractive alternate, including costs and influencing factors. The order-of-magnitude cost estimate developed during this phase is used to establish the project budget.

Layout and planning for the facility should include the following elements:

1. Environment

Chapter 13, Environmental Engineering, AREA Manual for Railway Engineering discusses environmental considerations in detail. Environmental factors to be considered include:

a. *Air*
TOFC/COFC facilities cause only minimal air pollution when asphalt or concrete paving is used. Exhaust fumes from motor trucks and other equipment are the major air pollutant.

b. *Water*
Water pollution is usually caused by fuel spillage at service and shop facilities.

c. *Noise and Lighting*
Noise pollution and nighttime illumination are the most serious operating problems, therefore the site should be located in thinly populated or industrial areas if possible.

d. *Rainfall Runoff*
Retention ponds are sometimes necessary to regulate the flow of water into streams or sewers due to the increased runoff.

e. *Archaeological and Historical Sites*
Archaeological and historical investigations are also necessary in certain areas.

f. *Housing Displacement*
A minimum acquisition of houses is recommended to minimize community disruption and project cost.

2. Economics

The ideal facility topography is relatively level with good cross drainage and stable foundation material. Heavy grading seriously affects the economy of the site. The site size and shape influence the facility design, which will affect operational efficiency. The site should be selected using through train pick-up and set-out, or termination and origination where possible. A minimum of switch engine moves should be used to assure the most economical return. The terminal should be located as near as possible to the center of the area where the shippers and/or receivers are located. Such improvements will tend to extend the range of profitability to shorter distance hauls.

3. Traffic Volume

Projected traffic volumes are determined before beginning the site-selection planning. The anticipated average and maximum volumes influence layout and the traffic circulation plan. If some of the trailers or containers are through-traffic and remain on the flat car, consideration should be given to providing set-out tracks for this traffic.

4. Size

The size of a terminal depends on the number of trailers/containers loaded and unloaded in a specific time period, the length of time the trailer/container is held at the facility and the method of operation. Some facilities are planned to store trailers/containers that are not involved in the day-to-day operation of the terminal. Space is often limited and expensive, which therefore leads to more mechanized storage and handling systems. To determine these space requirements, future-volume projections should be obtained for both loaded and empty trailers/containers storage.

5. Standardization

Standardizing certain elements of a TOFC/COFC terminal is desirable. It has the advantage of similar operation, equipment and conditions at each terminal. It provides a safer operation, lessens inventories of spare parts for equipment and reduces cost by the purchase of large quantities. Standardization of equipment and methods permits transfer of equipment from one terminal to another.

6. Highway Access

Good highway access is essential to the proper TOFC/COFC terminal siting. Highway load restrictions and bridge clearances must be considered.

7. Rail Access

The approach tracks should be free from rail traffic congestion and have the proper rail clearances.

8. Zoning

Most governmental jurisdictions have zoning laws and it is important to determine if the proposed site is zoned to permit constructing the facility. Land can be rezoned, of course, but this is usually a lengthy process.

II. FACILITY TYPES AND EQUIPMENT

Committee 6, Assignment 1 entitled "Buildings, Platforms, Ramps, Paving, Lighting and Other Facilities for Piggyback Terminals," in Bulletin No. 625, Proceedings Volume 71, January 1970 discussed mechanical loading considerations, loading methods and yard design considerations.

There are three types of TOFC/COFC facilities. . .end, side and overhead loading and unloading. . .and each has different cycle times.

A. General

Certain support activity components have static time values regardless of the equipment used. The cycle time for support activities is about ten min and includes such functions as driving from the gate to the parking lot, dropping the unit, driving from the parking lot to the track, dropping the unit at the track, picking up the unit at the track, driving from the track to the parking lot, hooking the unit and driving from the parking lot to the gate.

The approximate cycle time for each method during typical TOFC/COFC loading/unloading operatons is five min for end-loading; two and one-half to three min for side-loading (TOFC); and one and one-half to two min for overhead loading (TOFC). There is really no appreciable cycle time difference between sideloading and overhead loading. An additional five to seven min are required when putting the container on a chassis.

B. Volumes

Three ranges of TOFC/COFC facility volumes are discussed and are defined as the total number of trailers or containers loaded or unloaded in each 24-hr day.

1. Low Volume—Less than 100 Lifts/Day

Low volume terminals are characterized by infrequent train service. Trailers are delivered or picked up by a local drayage or cartage agent and are placed on or removed from a flat car by a hostler using a fixed or portable ramp. Some parking or yard space is necessary if inbound trailers must be unloaded before outbounds can be loaded. In many cases, trailers can be ramped as they arrive. Container operations occur at low-volume terminals only as containers-on-chassis unit movements, and the unit is considered to be just another trailer to be loaded or unloaded.

2. Medium Volume—100 to 300 Lifts/Day

Medium volume terminals are designed to maximize the availability of trackside parking. Ramps can be used although cranes and side-loaders are preferred.

3. High Volume—300 and More Lifts/Day

High volume terminals are distinctly different from low- and medium-level terminals in configuration, equipment and method of operation. These facilities use multiple side-loaders or crane-loading equipment. Rail-mounted cranes can be used, but the majority use cranes mounted on rubber tires.

Some terminals will handle only containers on a chassis and others will have a separate storage area for ground storage. When a trailer enters the terminal area, the trucker is directed to leave the trailer in a specific parking area. Hostlers then pick up trailers for spotting at trackside before loading operations begin.

C. End-Loading (Ramp, Fixed or Portable)

Railroad cars are end-loaded by backing the tractor-trailer combination on a flat car or string of cars using a platform or ramp constructed at car-floor height. The reverse procedure is used to unload trailers. The tractor is equipped with a hydraulic fifth wheel and usually is identical to the jockey tractors used at motor carrier terminals. The normal crew size for this method consists of one tractor driver and a tie-down person; however, two tie-down persons are used at certain facilities. This is a popular method of operations for several reasons:

- Low initial investment for facilities and equipment
- Economical in low-volume locations
- Ability to handle up to ten 90-ft cars in a string
- Low maintenance cost

The major shortcomings of this type operation are:

- Inability to select loads to be ramped or deramped; trailers must be handled as they appear in the consist.
- Potential for delays in bad weather due to slippery surfaces on railcars.
- Inability to handle containers without chassis.
- All trailers must be facing same direction for unloading a string of cars. With single-end ramps, inbound flat cars must be taken to a wye track or a turntable for turning, which is often time-consuming and costly.

D. Side-Loading

Side-loading and unloading can be done by a fork-lift truck, a platform at car-floor height, a depressed track or by special equipment which permits separating the trailer body from its wheels and placing the body on a flat car.

Most side-loaders can lift from either the top or the bottom. Side-loaders pick up the trailer from trackside, move forward to the flat car, lower the unit on the flat car and release it. Picking up a container from the bottom must be done with a tractor and bogie, or chassis. The chassis must be removed from the position near the rail car as the container is picked up so the side-lift unit can move closer to the side of the rail car. The transfer unit can then move up to place the container on the rail car. Following placement, it then moves on to the next unit. The unloading operation is the reverse procedure of the loading sequence. With the ability of picking up from the top, the container can be on the ground, stacked on other containers or on a chassis. The transfer unit brings the container to the track and places it on the rail car. Unloading is the reverse procedure.

Side-loading operation requires either two or three people in addition to the hostler, depending on the location. One person operates the side-loader and the others are on the ground to guide the machine operator. In a container operation, the people on the ground also help secure or release the container from the chassis and flat car.

Side-loader characteristics vary in accordance with the manufacturer. The following list displays key information regarding the major types of side-loaders now in use:

1. Capacity: 44,000 to 90,000 lb
2. Minimum aisle: 30 to 55 ft
3. Ideal aisle: 55 to 75 ft
4. Turning radius: 20 to 52 ft
5. Transport to storage area: Yes
6. Speed: 9 to 26 mph
7. Stacks containers

Side-loading has gained popularity in recent years because of the following operating advantages:

- Flexibility to pick "hot loads" in the middle of the train.
- Ability to handle both containers and trailers.

Side loading has the following disadvantages:

- Equipment is expensive compared to ramps.
- Equipment has a wide turning radius, requiring a substantial amount of maneuvering space.
- Equipment has poor weight distribution which increases both the subsurface density and paving thickness requirements and, thus, greatly increases construction costs.
- Lost time due to equipment down-time and maintenance.

E. Overhead Loading

Gantry cranes permit overhead handling of containers and trailers. The cranes may be rubber-tired or rail-mounted, either of which permits, picking up trailers or containers from the roadway adjacent to the track and longitudinal movement from car to car. Rubber-tired cranes often require reinforced concrete pads to support the wheel loads, while a rail-mounted crane requires a firm foundation to support the crane rail. The cranes may be further classified by the type of lifting system. A top-latch system lifts the container with extended twist locks which are inserted into holes in the corner castings on top of any International Standards Organization container. A bottom system is characterized by its straddle arms which lift the container or trailer

from underneath. Many cranes have dual lifting capabilities and may also alter their capabilities by varying the gantry span so as to serve two or more tracks and roadways.

In an overhead crane operation, a trackside parked trailer or container is lifted vertically and moved laterally to the flat car, and lowered onto the car. For the most part, the major difference between a container and a trailer operation is in the greater precision required in spotting the crane and the container so as to properly engage the rail car latches into the container corner castings. Ground operations supporting container loading/unloading operations are more complex because the bogies or chassis must be brought trackside.

At least two people are required for overhead crane operations—a crane operator and a tie-down person. The tie-down person ties and releases the trailers to/from the stanchions and directs the crane operator. Two to four tie-down persons may be involved in a container operation to operate the twist locks on the corner castings to secure or release the container from the chassis or flat car.

The characteristics of gantry cranes vary in accordance with the manufacturer. The following list displays key information regarding the major types of gantry cranes now in use:

1. Capacity: 50,000 to 100,000 lb
2. Span (rubber-tired): 32 to 76 ft
3. Transport to storage area: possible with rubber-tired units.
4. Stacks containers: four high (maximum; however, not normally desired).
5. Turning radius: generally five ft over gantry width.
6. Number of lifts per day: 360

This system offers the following advantages:

- Flexibility to pick and choose the loads.
- Ability to unload on one side of the track and load from the other.
- Capability to span two tracks at once, unloading or loading from both. (Using a special long-span crane.)
- More stable operation than side loading because of better weight distribution.
- For rubber-tired cranes, flexibility to travel within the yard.
- Selective loading and unloading.

Overhead loading has the following disadvantages:

- Not as mobile as side loading (cannot pick units out of the parking area).
- If rail-mounted, cannot switch from one track to another.
- Inefficient when spotting tracks holding less than 20 cars.
- More expensive equipment than ramps or side loaders.
- COFC operations less effective because of the limited area for stacking and removing containers.
- Loss of time due to equipment down-time and maintenance.

F. Parking/Staging Equipment

1. Containers

a. *Yard Tractors*

Yard tractors include flatbed trucks and trailer/chassis units which move containers from trackside to the parking area. Once in the parking area, the container may either be left on the chassis or flatbed or may be removed and stacked by a fork-lift or travel crane.

b. *Straddle-Carriers*

Straddle-carriers are specialized units designed to transport one container at a time between trackside and the parking area.

The unit is usually operated when the container is located on the ground, although it can lift containers on or off a chassis. Further, most models cannot stack containers over two or three high.

c. *Fork-lifts*
Heavy-duty fork-lift trucks are used for stacking and repositioning containers. The effective stacking limit is two high. Slats are used to separate some containers for fork removal. These units are used to transport containers from trackside to parking areas, and some can also top-lift container.

d. *Travel Cranes*
Travel cranes are mounted on rubber-tired wheels or rail tracks and can straddle widths over 75 ft rubber-tired or 200 ft rail. They top-lift containers and can stack them up to four high. Travel cranes are most typically used to stack rather than transport containers, but they are not designed for flat car loading.

2. Trailers

Trailers are usually moved from trackside to the parking area by a yard or road tractor.

G. Standard Rail Cars

The type of railroad cars to be used in the facility must be considered in the design. The 89-ft flatcar is the normal car used for TOFC/COFC service. These cars will accommodate two 40-ft trailers or containers. The 45-ft box is rapidly becoming standard for trailers or containers; the 89-ft flatcars generally cannot handle two of these. This limitation reduces the efficiency of 45-ft trailer movement via TOFC. Track length should be designed on total car length, draw bar to draw bar.

H. Special Intermodal Cars

Various kinds of special intermodal cars have been studied to reduce weight, improve aerodynamic efficiency, reduce fuel consumption, reduce the number of locomotive units needed to move a given consist and improve terminal operations. Current research and development in these areas is very active. With the renewed emphasis on intermodal service and new cars which must be machine-loaded, terminal modernization becomes a must. TOFC/COFC facilities designers should review the status of these special intermodal cars and accommodate the most promising special intermodal cars in future designs. An important aspect is that a car design must pass the required tests if the car is to be certified for unrestricted interchange. Some of the types of cars currently planned, being tested, and/or in revenue service are:

1. Articulated skeleton cars for trailers and containers.
2. Dual-mode vehicles (rail/highway vehicle).
3. Articulated double-stack container cars.
4. Articulated well-cars, some with a mix of trailer/container-carrying capabilities.

I. Trailers

The size and weight of truck trailers operating over highways are controlled by state and federal law. The most common trailer size is 8′ × 9′-6′ × 40′ with an 80,000-lb gross weight restriction for a tractor-semitrailer combination. The surface transportation assistance act of 1982 and the DOT appropriations act of 1982 authorizes vehicles up to 80,000 lbs gross weight; trailers up to 102 in wide and semi-trailer portion of a tractor-semi trailer combination up to 48 ft long on sections of the federal aid Primary System highways. Applicable current state and federal legislation should be checked. The allowable load limits and the seasonal weight restrictions on the access roads to the TOFC/COFC terminal are important, possibly restrictive design criteria.

J. Containers

Essentially all of the containerized shipments transported by the railroads today are in maritime containers and this trend will probably continue. Although the container sizes have not been standardized, the most common sizes are is 8′ × 8′ × 40′ and 8′ × 8′ × 20′.

K. Tie-Down Mechanisms

Two types of retractable tie-down mechanisms or trailer hitches on flatcars are presently in use, the "wrench-operated" and "tractor-operated". The wrench-operated mechanism requires a special wrench (usually electric or pneumatic) which drives worm screws to raise or lower the mechanism, and locks or unlocks the fifth wheel pin. The tractor-operated mechanism for end-loading is pulled up from its retracted position by the tractor that is used to load the trailer onto the car. During unloading, the tractor is also used to "knock down" the hitch.

L. Chassis

Container chassis get very little maintenance, as a general rule, which results in a large number of damaged or unusable chassis at any one time. Chassis damage is less likely, however, than trailer damage from dents and cuts on the compartments. Motor carriers are often faced with the alternative of accepting a damaged chassis or transporting a container on a flatbed trailer. The damaged chassis creates conditions which may violate state laws, cause equipment or cargo damage or breach insurance policy conditions. Transporting containers on flatbed trailers requires using chains to secure the container and additional labor to perform the tie-down. Chassis-storage provisions should be made in the designs for all terminals that will handle containers.

III. DESIGN FACTORS

The design factors that must be accommodated include the type of terminals, layout and configuration, the pavement systems, parking and storage, security, facility services, the terminal buildings and the maintenance and service buildings.

A. Terminal Types

The facility should be designed so the truck driver can check in at the gate and then park the trailer in a well-marked parking area. Truck drivers picking up inbound trailers should likewise be directed to parking areas. The advantages of drivers parking or picking up trailers at trackside are great, but this often cannot be done because of the driver's lack of knowledge of trailer spotting and the untimeliness of customer pick-up. Labor agreements sometimes prohibit parking at trackside.

1. End Loading

The ramps on a stub-end track can be constructed from timber, steel, or concrete filled with earth. Track for two-directional unloading can be provided by paving an area level with the top of rail on which a portable ramp may be maneuvered. Portable ramps can be used to eliminate the need to turn cars. The unloading track must be tangent. The maneuvering area at the toe of a single ramp should be about 75 by 100 ft. Between-track platforms or platform walkways adjacent to tracks are desirable for end-loading and unloading facilities. These platforms, which permit easy worker movements between cars should be about 2 ft-3 in wide. The recommended height is 3 ft-6 in or car floor-height. Laws governing track clearances affect the width of these platforms. Power-wrench outlets supplying electricity or air should be considered between tracks, although portable power sources can be used.

A small efficient low-volume end-loading terminal with the configuration shown in Fig. 1 can handle 30 to 50 trailer loadings per shift. The configuration will vary in accordance with the land available. The most important consideration is to locate the parking area as close to the ramp as possible and to provide a circulation pattern that will not interfere with incoming and outgoing trucks and with ramping operation.

2. Side Loading

The track and adjacent parking configuration shown in Fig. 2 with a length of five to ten cars can easily handle a volume of 50 lifts per shift. This low-volume configuration can be expanded for higher-volume terminals; up to 300 lifts per day can be attained with multiple-shift operations. This configuration permits loading on one side of the track and unloading on the other side of the track. As volumes begin to reach this capacity or other operational conditions indicate a two-track arrangement is required, the single-track facility can be readily expanded as shown in Fig. 3. A second track is added about 110 ft from the first track, with its own adjacent parking area outside of the track area. One-way traffic flow should be provided to minimize interference with the load/unload operation.

Traffic control and communications become very important when a medium-volume terminal approaches a volume of 300 lifts a day. An efficient operation will therefore require road and parking lane markings with large readable signs to direct truck traffic and parking. When a truck driver enters the gate, these signs should clearly indicate where to locate or pick up the required trailers, identify inbound or outbound trailers and identify each parking area by number. If the train is formed in blocks, signs should indicate the block direction.

An alternative to the outside parking shown in Fig. 3 is a configuration with adjacent parking between the two tracks as shown in Fig. 4. The traffic-flow patterns are good with minimum interference with the side-loading operation. While this configuration requires more land, it provides a more efficient terminal where side-loading is used.

3. Overhead Loading

The crane loading-rate is somewhat faster than the side-loading rate. Therefore, replacing side-loader equipment with crane-loading equipment should be explored when lift volumes approach 250 to 350 lifts per day. The major advantage for initially constructing an overhead loading facility is that it requires significantly less land than a side-loading facility. Overhead loading is usually provided at high-volume terminals with 300 or more lifts a day. The high-volume terminal shown in Fig. 5 equipped with two cranes can be used for daily volumes in the range of 300 to 600 lifts a day. This terminal configuration can then be expanded from 600 to 1,200 lifts a day by adding tracks and cranes. The number of cranes required depends more on peak demand and travel time than on the total number of trailers it could conceivably handle in one day.

Expanding terminals to volumes of more than 1,000 lifts a day should be considered only after a very thorough analysis has been made of truck-traffic flow-patterns. In major cities where volumes of these magnitudes may be available, the efficiency of several high-volume terminals

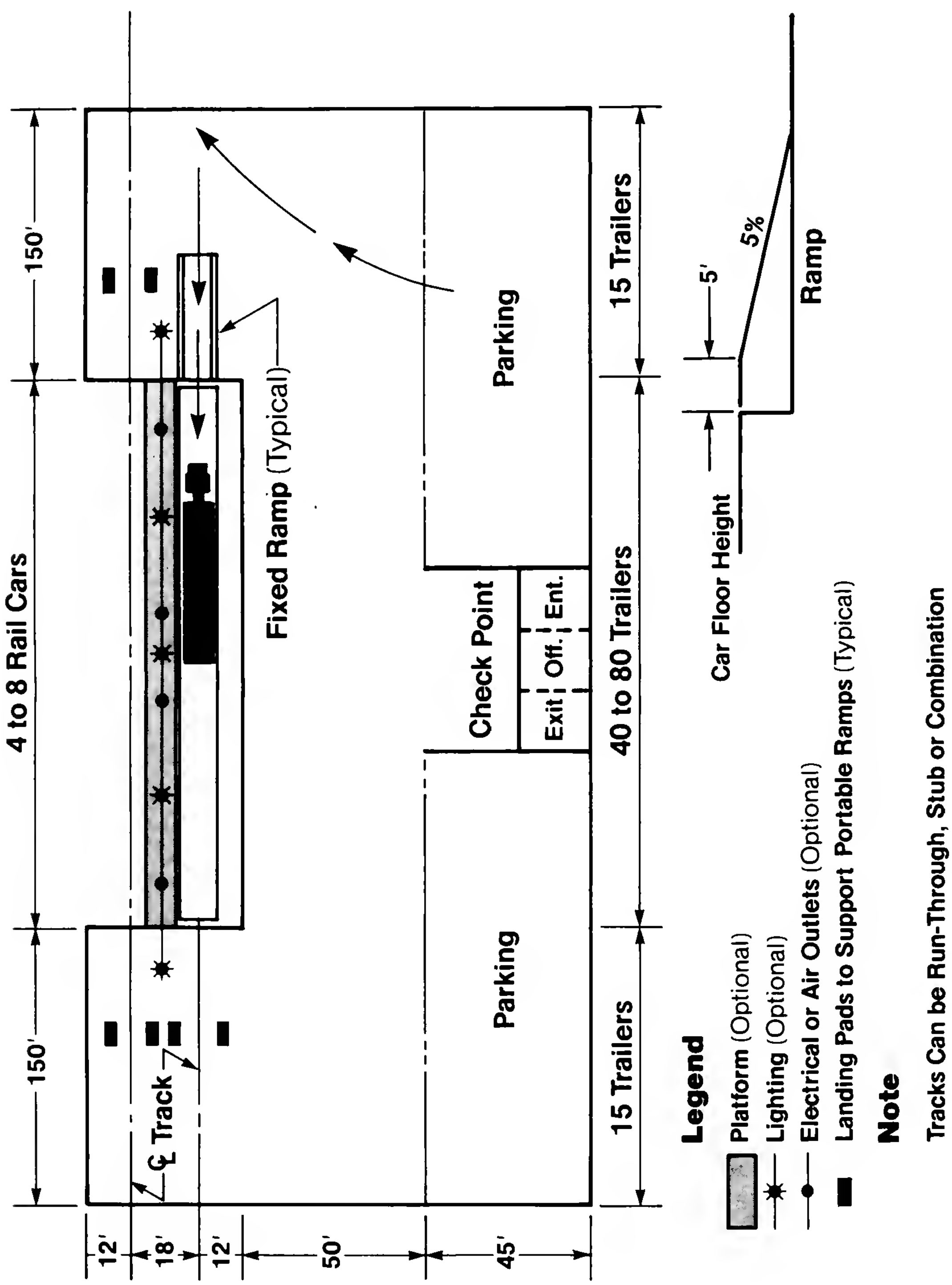


Figure 1—Low-Volume Terminal With End Loading

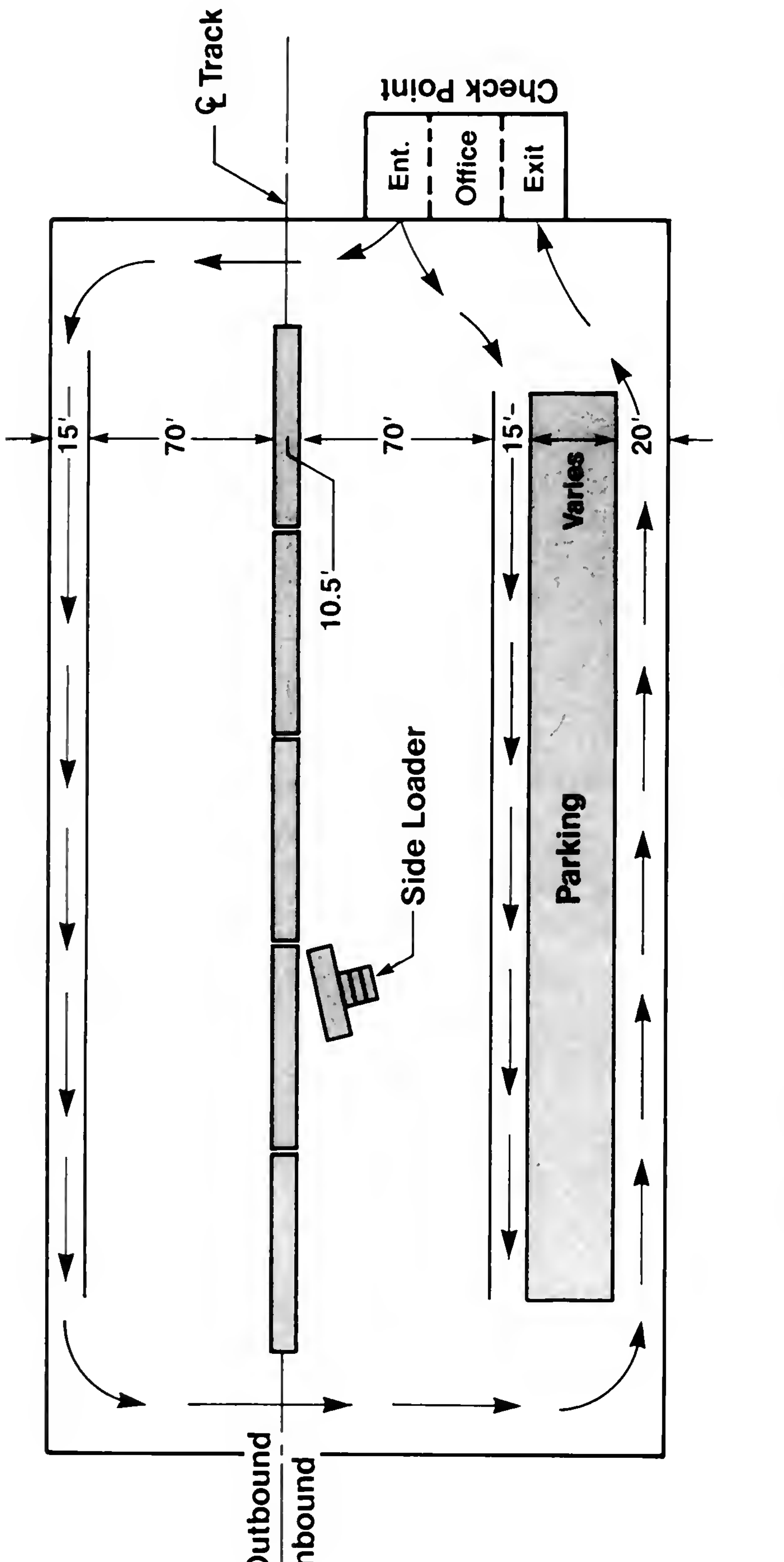


Figure 2—Low-Volume Terminal-Side Loading

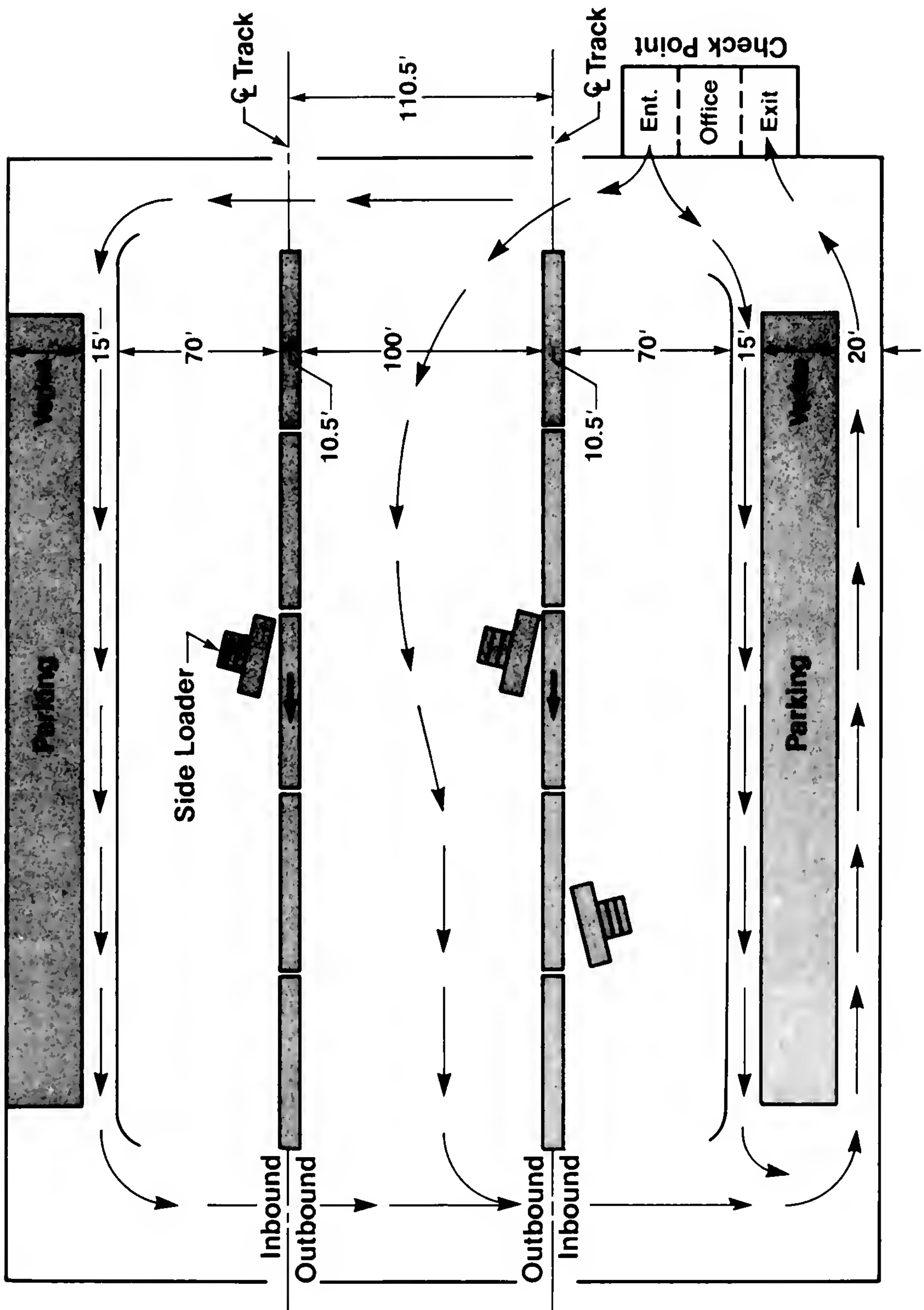


Figure 3—Medium-Volume Terminal With Side Loading and Outside Parking

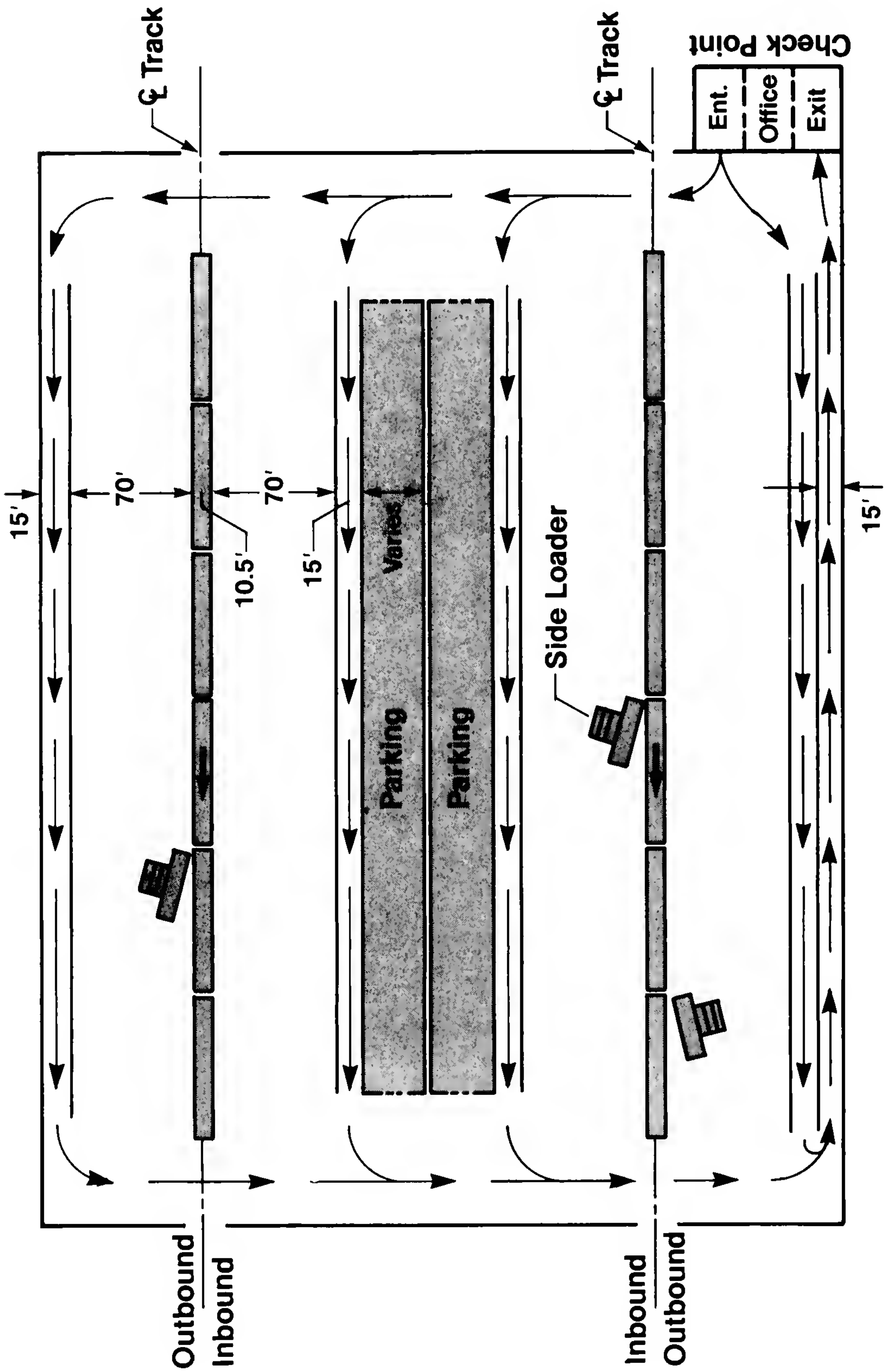
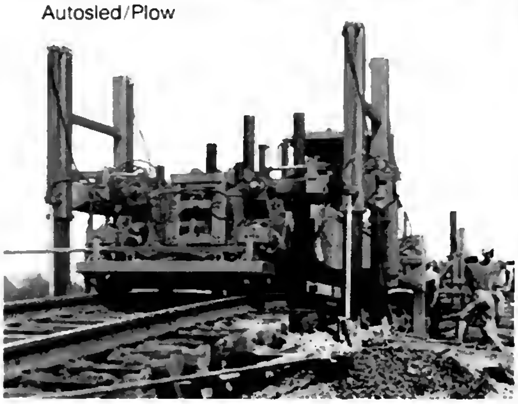


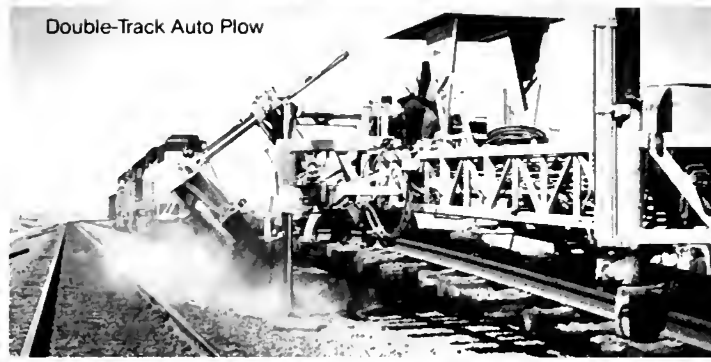
Figure 4—Medium-Volume Terminal With Side Loading and Inside Parking

Shoulder Ballast Cleaner

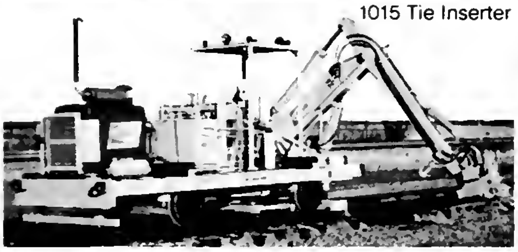
Winch Cart



Autosled/Plow



Double-Track Auto Plow



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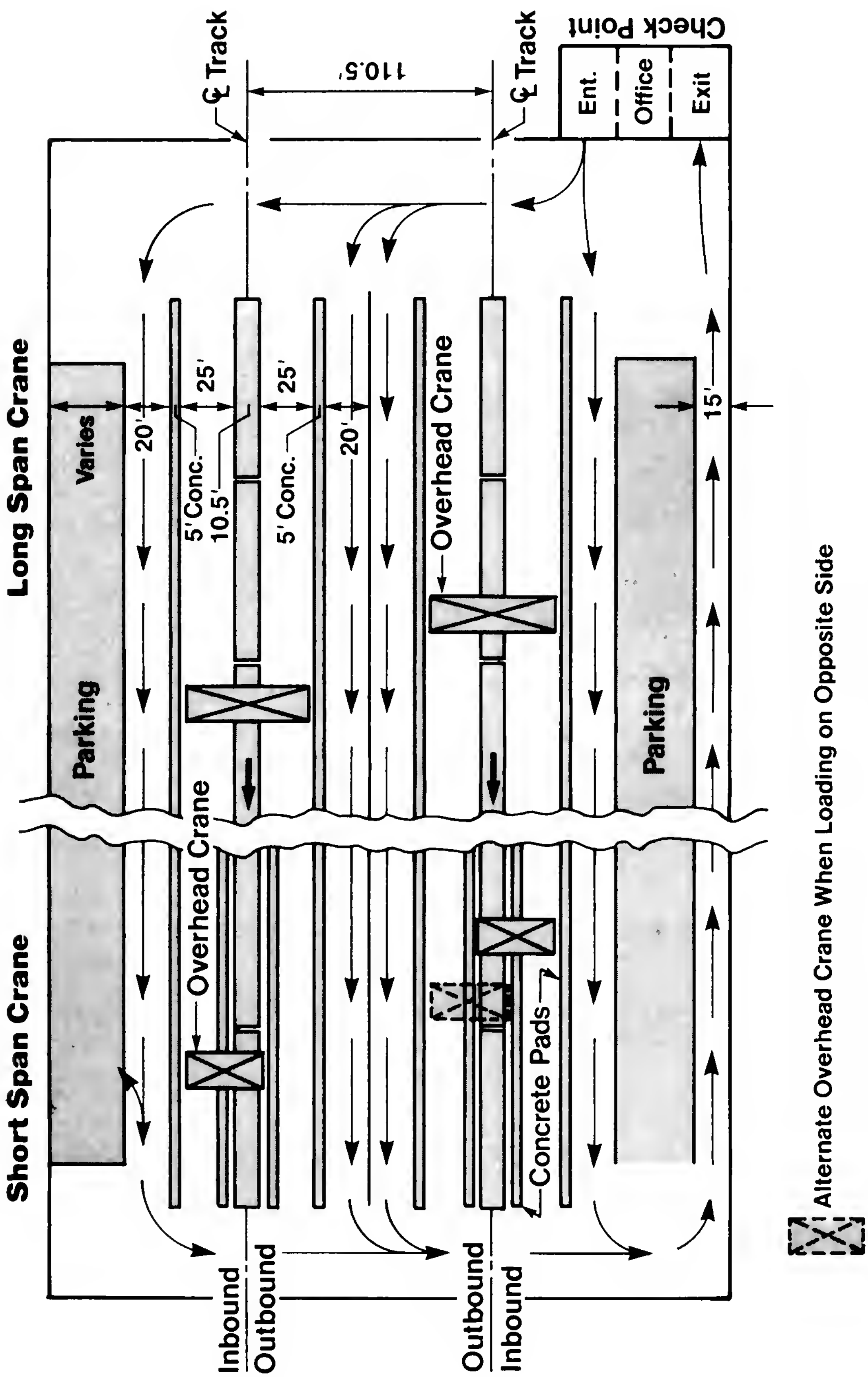

Figure 5—High-Volume Terminal, Crane Loading with Outside Parking

located at strategic points around the city should be contrasted with the efficiency of a single very-high-volume terminal.

B. Layout and Configuration

The type of loading-unloading equipment to be used in a terminal influences the terminal layout and configuration. The types of equipment available, including their space and other operating requirements must, therefore, be carefully studied and compared. The facility should then be laid out to conform with the requirements of one or more of the equipment types under consideration.

Terminal layouts also affect the efficiency of loading/unloading and parking activities. The number of railcar spots to be allowed for in the design of an end- or mechanical-type loading-unloading facility is determined by the volume of traffic and the availability of switching. Rail-mounted cranes require the least amount of lateral space but may complicate the problem of movement between rail cars. Consideration must also be given to the operating characteristics of any back-up loading-unloading equipment. While portable ramps are inexpensive, access room is required at the end of the rail cut. Side-loaders are flexible, but need more operating space between parallel tracks. The requirements of a combination TOFC/COFC facility should be considered when determining the most effective equipment and layout.

The most common configuration for medium-volume terminals consists of multiple parallel tracks with the appropriate space between each set of tracks for equipment operation. The tracks vary in length from about 1,000 to 3,000 ft and are usually stubbed although some facilities have flow-through trackage.

High-volume terminals are similarly designed, but have typical track lengths of 3,000 to 4,000 ft with a driveway crossing near the middle for ease of trailer handling by yard hostlers. High-volume terminals can handle up to 1,000 units per day flowing through the facility. The typical high-volume terminal does not have the track capacity needed for a full day's volume of rail car traffic and cars must be pulled into or out of the facility several times a day. The terminal layout can affect the number of pulls required as well as the time required to make them. With through-tracks, road crews can do some or all of the spotting.

There may be advantages in having the track area depressed below the parking area, driveways and ramps. This is not commonly done, however, because of track maintenance and car inspection and repair problems. Either depressed tracks or elevated platforms will bring the railcar floor to the same level as the adjacent loading area and permit a roll-on/roll-off operation. The tracks used to spot car for loading should be tangent. The approach track curvature should be as flat as possible, but limited to 420-ft radius.

The space between the tracks should be adequate to operate cranes or side-loading equipment.

C. Pavement Systems

1. General Considerations

There are three primary requirements for pavements at TOFC/COFC terminals:

a. the capacity to support parked loaded semi-trailers and containers;
b. the capacity to support lifting equipment while carrying and lifting maximum loads. (Side-loaders in particular have heavy concentrated front-wheel pressures when under load and on the steering wheels when empty.); and

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c. the pavement should require minimum maintenance because closing part of a terminal for such work can seriously impair service to customers as well as cause difficult operating problems.

Economy in capital expenditures is always an important consideration and particularly so in TOFC/COFC areas where the projected economic life of paving is short, possibly ten years or less. Some of the factors causing short paving life are potential modifications in loading-unloading equipment that can permit the operation to be done by one man and/or reduce (if not eliminate) hostling with yard tractors; variations in the trailer/container proportions; loading or unloading railcars carrying two tiers of containers; and the impracticality of designing a pavement with enough flexibility so it can continue to be used in the event such changes are made.

2. Recommended Design Practice

The paper in *Bulletin 633* (Proceedings Vol. 72, June-July 1971) entitled, "Design of Pavements for Container Handling Areas" by F. L. Peckover and W. W. Wong and the ensuing discussion papers that followed it in *Bulletin 636* (Proceeding Vol. 73, Jan-Feb 1972) should be carefully reviewed. Information should be obtained from auger borings or other methods to identify all soil strata to a depth of about 10 ft below final grade. Ground water levels and frost depths must be determined and the drainage characteristics of the various soil types should be assessed. Where asphaltic-concrete surface courses are proposed, the maximum wheel loads, tire pressure and the California Bearing Ratio of the subgrade and subbase must be found. The data needed for Portland cement concrete pavement includes the maximum wheel loads, tire pressure, the modulus reactions of the subgrade and subbase and the 28-day flexural strength of the concrete.

The maximum grades recommended for a semi-trailer storage area are from 0.5 to 1 percent. Steeper slopes may overcome the initial rolling resistance of trailers with landing gear supported by steel dolly wheels and parked with brakes not set.

Access roads should have a minimum of two 12-ft lanes with a minimum 5-ft shoulder on each side.

3. Portland Cement Concrete Wearing Surface

This type of wearing surface is often used for runways for operating fixed-path equipment such as rubber-tired gantry cranes. Runway widths vary from 5 to 10 ft. Thickness will depend on factors given in the references mentioned above in paragraph C.2. Reinforcement is not required if the maximum longitudinal and transverse joint spacings given in these references are not exceeded. Outside edges thickened 25 percent for a distance of about 6 ft inward are recommended if the pavement will carry front-loading vehicles. Thickened edges are unnecessary for runways for fixed-path machines which are driven off them only at the ends or at one or two intermediate track crossings.

Reinforced concrete pads for the semi-trailer landing gear are recommended. The pads should be at least 5 ft wide and 8 in thick. However, soil conditions should govern. The center line of the landing pad should be 10 ft from the front of the parking stall or trailer.

4. Asphalt-Concrete Wearing Surface

This paving is frequently used for trailer/container parking space and for truck driveway and maneuvering areas. Its thickness, and that of the underlying base and subbase layers, should be determined from the criteria mentioned above in the references in paragraph C.2.

Where a crushed-stone base course is specified, graded aggregate up to 3 inches is satisfactory. For subgrades having a CBR value above 5 or on the border between values of poor and fair, the minimum base-course thicknesses for truck driveways and parking areas is 8 in of crushed stone or, alternatively, about 6½ in of an asphalt-concrete-base mix. An asphalt-concrete surface course having a minimum thickness of 1½ in base could be applied over either base course. In hot climates, the use of larger aggregate will reduce rutting.

5. Asphaltic Surface Treatment

This type of paving can be constructed where economy in initial capital expenditures is vital, yet a dust-free surface is necessary. At least two applications of emulsified asphalt is recommended over a 10-in minimum-thickness stone base course.

6. Waterbound Macadam

This is the least expensive type of paving and can give statisfactory service at small- to medium-size ramp-loading terminals. Dusting is a problem, its surface will not accept parking stall or other traffic markings, and the maintenance costs will be high. A minimum 10-in thickness is recommended.

7. Specifications and Construction Procedure

Individual state highway department standard specifications can be used for constructing the pavement systems.

D. Parking

Parking facilities should be near the loading tracks with additional parking for storage, as required. Access should be limited with a check-in and check-out lane for cartage companies.

1. Trailers

Yard and road tractors are the primary method of moving trailers within a terminal. The arrival of 500 trailers a day at the high-volume terminal requires giving special consideration to the parking problem. Ideally, parking will occur at trackside to minimize the necessity for additional labor to move the trailer. This may not be feasible, however, because of limited trackside space or other considerations. Preassigning all track-side spots is often necessary, even though the train schedules often do not coincide with the time trailers are brought to the yard for shipment. Special attention, therefore, needs to be given to the size and nature of parking space, its location and the manner in which trailers will be spotted or organized as they reach the terminal.

a. *Layout and Configuration*

The trailer parking configurations shown in Fig. 1 through 5 should be adequate to accommodate the daily traffic in each type yard. If the terminal will have a large number of stored trailers, additional parking area should be provided near the facility. A 10- by 50-ft area should be provided for each trailer.

When fencing is used, parking blocks or reinforced guard rail should be placed about 10 ft from the fencing for use as a wheel stop, giving consideration to the method of parking arrangements. If trailers are to be cleaned, additional space should be provided between the trailer and the fence for cleaning equipment.

b. *Capacity*

A trailer parking area of approximately two-and-one-half times the number of trailers handled each day should be planned. Consideration should also be given to the number of trailers that will be stored at the terminal.

2. Containers

A well-organized storage area should allow for presorting containers to facilitate loading in

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the order of delivery or shipment. Containers are stacked on the ground by both the straddle-carrier and the travel crane. The crane is used to move the container from the chassis or flat bed pulled by a yard tractor. Alternatively, the containers can be left on the chassis. Some railroads use flatbeds only in emergencies.

a. *Layout and Configuration*
There are three basic configurations for parking/stacking the containers. Each configuration affects the required space and the flexibility and ease of access to the containers. The configurations are:

1) *Herringbone Layouts*—used to store containers either on chassis or on support legs, with no stacking. Ease of access to any container is therefore excellent, but the space required for this layout can be prohibitive for a large operation.

2) *Block Layouts*—the best use of scarce parking areas; containers are stacked three or four high in a tight block. Selectivity in reaching a particular container can be seriously reduced in this layout. The layout lends itself best to storing containers which are stacked in a pre-determined loading pattern.

Block layouts are often used for storing empty containers and for long-term storage of containers awaiting outbound movements. Locating a container tends to be complicated since unit numbers are not easily visible.

3) *Ribbon Layouts*—offer better container selectivity than block layouts. Although stacked three-to-four high, access is maintained to at least one side of each container. Significantly more space than block layouts is required.

b. *Capacity*
A large number of containers are loaded or unloaded at marine terminals in a short period of time and the storage-yard capacity will control operational efficiency. The number of containers handled at inland terminals is usually low, but consideration should be given to the separate storage of containers without chassis and trailers.

E. Security

TOFC/COFC facilities are easy targets for both organized and random burglary. The facility must be physically open to allow entry and departure of trains. Most trailer movements are controlled by transit drivers who are not terminal employees. Further, the terminals are often in high-crime environments where stealing operations can be highly organized. Security for TOFC/COFC facilities is therefore essential.

Measures to reduce cargo losses include fencing the entire facility with adequate guards at the gate and patrols. Adequate facility lighting is necessary. Other methods to limit cargo losses are a closed-circuit TV to scan the yard and other critical areas and an electronic system with a small sensor to detect any unauthorized door or trailer motion. The sensor will automatically send an alarm signal to a designated location via radio and land lines and to the roving police officer via a small portable radio.

As the trailers enter and leave the terminal, the bill of lading must be checked and the trailer inspected for damage and/or broken seals. These procedures tend to decrease service capabilities at peak periods since they extend the time a trailer/container is at the terminal.

Reducing the check-in and check-out congestion at terminal gates can substantially improve the service characteristics of the entire operation. In high volume terminals, a separate security gate should be provided to check ID of driver and vehicle, and to make certain that driver/vehicle are at correct terminal. Main gate is used to complete transaction for transfer of cargo, weighing, etc. In high and medium volume terminals, main gate should be designed to provide optimum number of lanes needed at peak times. Since inbound peaks rarely occur at same time as outbound peaks, the individual lanes should be reversible to avoid overconstruction. When security and main gates are used together, processing time through main gate is longer than that through security gate. Therefore, fewer security lanes are required, and holding area must be provided between gates.

F. Facility Services

1. Electrical

Lighting and power outlets in the track area can be provided to assist tie-down operations. Parking areas should be lighted if there are extensive night operations.

Desired lighting levels can be attained by correctly spacing the selected light standards. Usual design procedures and criteria are published in the Illuminating Engineering Society's *IES Lighting Handbook.*

Between-tracks illumination for end-type loading and unloading is recommended to be 5 fc at car-floor height and 10 fc at the tie-down wrench. The recommended lighting for parking and general track areas should be 1 to 2 fc and should be 10 fc for trackside and ramp maneuvering areas.

2. Communications

Communication facilities within and beyond the operation area should be provided for efficiency.

3. Utilities

Sanitary, water, HVAC, electrical utilities and possibly engine block heaters should be provided in accordance with the facility requirements.

4. Grading and Drainage

A typical drainage system layout usually consists of a trunk line parallel to the tracks with lateral lines running under the tracks at about 200- to 300-ft intervals with catch basins between each track. Inlets should be located at all gutter low points and at any planned low points in parking areas.

Disposal of surface and subsurface water must be considered as essential elements of design and construction. The subsurface investigation program includes obtaining information on subsurface water conditions and a reliable determination of the ground water table. Historical data that may be available can be very useful to the designers.

Local, state, or national agencies may have drainage-design requirements and may specify certain design procedures. In the absence of any jurisdictional agency, county or state highway department procedures are suggested for use.

U.S. Geological Survery topographic maps and aerial drawings are useful for analyzing both topography and drainage, and show elevation contours, bodies of water, streams and other features from which the rates of water run-off can be established.

It is imperative to follow good engineering practice in all elements of design and construction, including preparing and compacting of the subgrade, increasing the bearing capacity of soils as necessary, erosion protection, proper sloping of cuts and fills and surface and subsurface drainage system.

5. Water Pollution Control

Water pollution control should be provided at the following servicing areas:

a. fueling;

b. maintenance building;
c. outside maintenance areas; and
d. trailer and truck washing facilities.

6. Truck Scale

A truck scale meeting state highway specifications may be needed to check the weight of loaded trailers. When required, it should be installed in the facility entrance/exit area.

H. Terminal Buildings

1. Offices

The larger-volume operations will require an office for supervisory and clerical staff, with the normal amenities for operating personnel. The size of office buildings will be determined by the number of employees. Standard office design criteria should be used, including provisions for employee parking.

2. Storage Building

A storage building should be provided for blocking and bracing material for adjusting shifted loads.

3. Air Compressor Facilities

Air compressor facilities are required for making brake tests on cars and for the use of air tools.

4. Interior Washing Facilities

Interior washing facilities and appurtenances may be necessary if refrigerator trailers are handled in sufficient quantity.

5. Container Transloading Building

A container transloading building may be required.

6. Guard Building

When the office is not located at the entrance, a separate guard building should be provided for check-in and check-out. Trailer roof inspections, when required, are made on trailers coming in and going out of the facility. There are several methods of making roof inspections:

a. Overhead mirrors;
b. High platforms with ladders or stairways; and
c. TV cameras monitoring from the interior of the office building.

7. Transfer and Customs Inspection Dock

A transfer and customs inspection dock may be provided for transferring loads from damaged trailers and for making customs inspections.

I. Maintenance and Service Buildings and Facilities

1. Locomotive and Car Maintenance

Maintenance operations for locomotives and cars at TOFC/COFC facilities are usually done at near service facilities. Locomotive maintenance may be provided at the site if the terminal is remotely located or if the operation is large enough to require its own facilities.

Much of the maintenance of both freight cars and locomotives is closely regulated by either the FRA or the AAR, and it is important to consider their repair procedures in designing a

maintenance facility. EPA and state environmental agency requirements must also be considered. Car repair at a TOFC/COFC facility normally includes light repair performed in the yard.

2. Trailer/Container and Tractor Maintenance and Servicing

The corner-post securing devices (for containers) and the fifth wheel/trailer securing devices must be periodically checked to ensure the devices are secure. This inspection is in addition to the normal rail car inspection procedures which railroads must do. In the case of refrigerated units, diesel-powered generators must be checked. Tractor maintenance may be provided when necessary.

3. Equipment Fueling Facility

A gasoline or diesel fueling facility for equipment should be considered. The equipment requiring fueling facilities are:

a. tractors;
b. refrigerated trailers;
c. gantry cranes;
d. side-loaders;
e. portable generators;
f. straddle cranes; and
g. other mechanical equipment.

4. Side-Loader/Crane Maintenance Facility

A separate building will improve maintenance of this equipment, especially in colder climates.

COMMITTEE 16—ECONOMICS OF PLANT, EQUIPMENT AND OPERATIONS

Report of Subcommittee No. 7 and I. R. Grunwell*

How Operations Research is Helping North American Railroads

C. Bach (Subcommittee Chairman); J. W. Rettie (Subcommittee Vice Chairman); J. B. Clark; D. O. Eisele; E. A. Gencarelli; M. N. Goodkind; A. M. Handwerker; J. P. Holland; M. C. Holowaty; S. M. Kiger; W. R. McGovern; R. T. Meyer; R. L. Milner; R. D. Penhallegon; D. A. Peterson; H. C. Petersen; E. J. Sierleja; H. Wanaselja; D. B. Weinstein

INTRODUCTION

This report examines the role of Operations Research in North American railroads today. Perhaps the best way to start is to define it.

Operations Research or Management Science as it is also known today, may be described as

*Project Officer, Operations Research, Canadian National Railways.

a scientific approach to decision making that involves the operations of organizational systems[1]. An application of Operations Research involves:

- Constructing mathematical, economic, and statistical descriptions or models of decision and control problems to treat situations of complexity and uncertainty.
- Analyzing the relationships that determine the probable future consequences of decision choices, and devising appropriate measures of effectiveness in order to evaluate the relative merit of alternative actions[2].

Operations Research had its origins in World War II when scarce resources had to be allocated in an efficient manner. Following the war, it developed in a number of industries, including the railroads. While not being railroad specific, this paper will look at the skills and the variety of work done by Operations Research groups within the North American railroad industry.

Organization

Operations Research analysis is being carried out by staff groups in railroad companies, consulting firms, and universities. External organizations perform the work on a contract basis to the railroad, or sometimes with funding from government agencies, e.g. the Federal Railroad Administration in the United States and the Transportation Development Center in Canada. In these latter cases, the work has benefit to the railroad industry as a whole. Of the Class I railroads surveyed for this report[3], one half of the O.R. groups are established within the Operations department while the majority of the other half is part of the Corporate Management Information Systems department.

Scope

Railroad Operations Research groups operate as internal consultants within the company. They vary in size from 2-30 analysts, with an average size of about 10 persons. The groups have considerable expertise in Operations Research, applied computer science, and other approaches to solving business decision-making problems. If a particular expertise is not available within the group, its relationships with other railroads, academic institutions and professional consultants usually insures that the necessary expertise can be acquired. Quite frequently, exchanges of information (especially computer models) are carried out between railroads to improve industry-wide expertise and avoid re-inventing the wheel.

Operations Research groups have undertaken studies in the Transportation areas of Operations, Marketing, Intermodal, Equipment and Engineering, as well as Finance, Personnel, Purchasing and Materials Management and Data Processing applications.

Skills and Techniques

Operations Research analysts possess a mixture of interdisciplinary skills. Academic backgrounds include university-level Bachelor and Master degrees in Engineering, Mathematics and Statistics, Business, Computer Science and Economics. Analysts have a broad viewpoint and a practical understanding of management problems. They must deal with everyone from long-time railroaders to computer specialists. To be successful, an Operations Research study must provide understandable, positive conclusions to the decision-maker when they are needed. Operations Research involves problem-solving, not research within an ivory tower.

Of all the skills and techniques used by an analyst, common sense is therefore the most important one; that, and an ability to filter all the available information, understand the problem, and formulate a solution using appropriate methodology to produce useful results. Problem-solving techniques used include statistical analysis, simulation, queuing theory, optimization, inventory theory, and forecasting. A requisite knowledge of computers is essential. Today, this also requires expertise in the area of microcomputers and their applications.

Activities

Operations Research work is carried out in all departments of a railroad organization. These activities frequently are divided into *applications support,* providing analytical assistance with models and techniques previously developed and refined, and *research and development* into unstructured and unsolved problem areas. Due to the nature of the business most of this work is being done for Rail Operations and Planning departments. This section discusses various applications, the savings, and impact that O.R. is making on the railroad industry.

a) Transportation

i) Line Planning

Today computers are an essential part of operating a railroad. Railroads could not stay in business and effectively compete without computer systems to maintain on-line freight car location and provide the support information for planning major business decisions. In B.C. times, (i.e. Before Computers), Transportation Planners did pen and paper calculations to evaluate the effects of plant and operational changes. This was arduous, tedious, and not always accurate. But with the advent of the computer, O.R. practitioners began development of operation models that enable Transportation Planners to make these evaluations more accurately, quickly and effectively.

Probably the first, and most often used operations model is the Train Performance Calculator (TPC). By simulating the acceleration, deceleration, and movement of trains across subdivisions, the TPC calculates minimum running times and tests the effect of various tonnage, power or line change configurations upon running times and fuel consumption. This is used to develop train schedules and determine the effect of Temporary Slow Orders (TSO's) on the Operation.

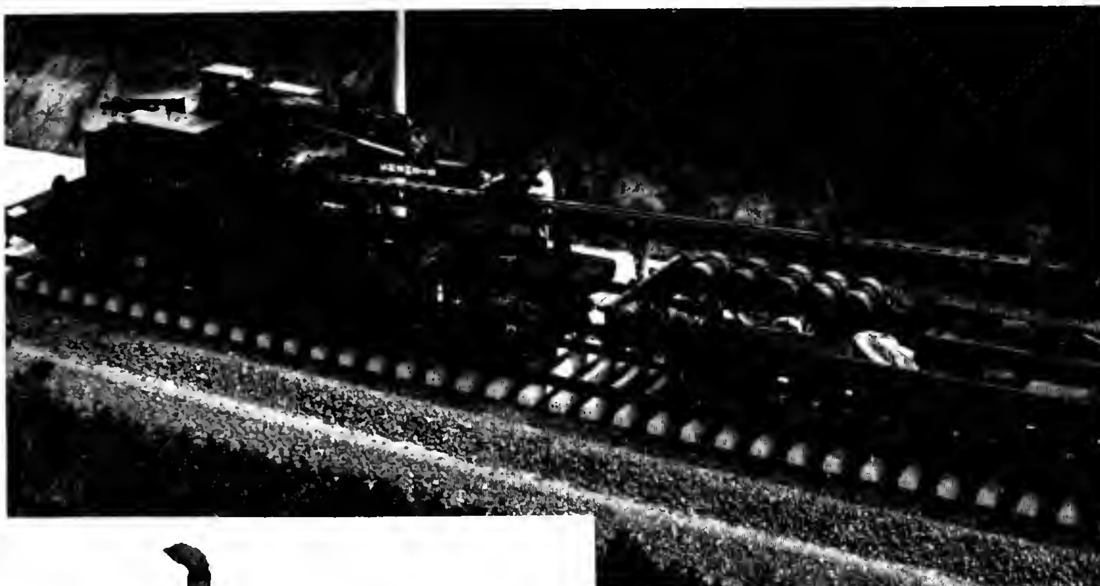
TPC outputs are an important building block for more sophisticated models. These models make line capacity decisions involving evaluations of single versus double tracking, siding location and length, signalling systems, and train dispatching strategy. These analyses involve the trade-off between capital investment and operating rationale. The resulting decisions govern investments of hundreds of millions of dollars in both plant (tracks and signals) and equipment.

The tools discussed above concentrate on the design and operation of the rail line on a subdivision level — but what about the system-wide effects of making changes to a part of the operation? This requires examination of the interdependencies of traffic priorities, train schedules, blocking policies, power distribution and terminal operations. There are many computerized network/service planning models that have been developed to answer these "what if " questions. These have been successfully applied by many railroads to rationalize traffic routings, terminal closings and even mergers with other railroads.

ii) Yards

Yards are a very important railroad resource since: a) one third of the total industry expenditure (approximately $4 billion) is spent on yard operations; and b) 62% of the total freight car cycle time is spent in yards.

There are two main concerns for yard planners—physical plant design/redesign and operational changes to existing yards to improve productivity. Traditionally these studies are carried out by yard survey/manual simulation methods. This approach is extremely manpower—intensive and expensive and provides a limited ability to examine operational alternatives. Today, computer simulations are used extensively for these purposes. The most sophisticated of these are interactive and use the yardmaster as the decisionmaker. In this way, these models have many uses beyond yard design including revision of train schedules, restructuring of operating plans and the training of yard staff.



New applications actively being pursued by individual railroads and the AAR are yard management aids such as Dynamic Track Assignment (flexible track assignment based on daily traffic demand) and Hump Sequencing (optimal scheduling of trains to hump) for implementation on real-time yard computer systems. The potential savings in the yard sector from this work in terms of reduction of operating expenditure and postponement of physical plant construction has been conservatively estimated in the tens of millions of dollars annually.

iii) Other Work

Operations Research analysts are involved in other transportation areas including track and train dynamics, derailment analysis, accident prevention, motive power fleet planning, and backshop scheduling. Improvements in each of these areas has a substantial impact on the overall railway operation.

As an example, consider motive power fleet sizing and purchase planning. Prior to the 1982 recession, planners were primarily concerned with making the correct selection of number and type of locomotives to maintain an adequate fleet size. This required evaluating on a regional basis the trade-offs between train speeds (i.e. weight/power ratio), allowable delay, and the size of fleet to be operated. At a cost of $1,000,000 per locomotive it is essential to maintain only the minimum required fleet. These analyses have been made using optimization/simulation models (typically Linear Programming techniques) to determine the "minimum cost" (both capital and operating) fleet size.

In recessionary times however, the questions have become not what to purchase, but, what locomotives should be stored considering such factors as the traffic volume, backshop scheduling, and the long-term maintenance cost. Adaptations of previous planning models have been made to answer these questions. Quick and correct decisions on storage can make a significant contribution to the difference between a loss and break-even for a railroad in tough times.

b) Marketing

A variety of statistical analyses and modelling techniques assist marketing managers. These include forecasting models, revenue and market share analysis, pricing studies and profitability analyses. Computer graphics are being used more and more frequently as an important visual aid in displaying the results of these studies.

An important contribution to the Marketing function has been the development of computer models to forecast freight demand. These have been linked with gravity models to predict the empty return movements. This information is essential to a railroad for budgeting and planning long-term strategies and equipment requirements.

Pressures from deregulation and the continuing recession are forcing Marketing to adopt new methods of responding to an increasingly competitive environment. In the past lengthy and complicated tariff schedules were submitted to regulatory boards for approval. Now a railroad must respond quickly to competitive maneuvres from other modes of transportation. Economic pressures have caused the cutback of uncompetitive means of transport and even more competitive pricing tactics. In response to this environment, the railroads are employing models to determine why shippers chose certain modes of transportation in preference to others. Also microcomputers are being used to keep track of the most recent rate increases and to identify rate inconsistencies. Large scale projects are underway in both Canada and the USA to automate rate retrieval and tariff updating through on-line databases. These efforts involve the cooperation among the private railways, railway associations and government agencies to store the millions of rates currently in effect in North America. Once these databases are available railways will be able to more carefully scrutinize the profitability of their traffic. The wide availability of interactive software makes it possible to perform what-if analysis of certain critical traffic. With these tools it is possible to examine the impact of such hypothetical questions as what is the impact of a 10% increase in rates on the profitability of traffic between two areas and achieve a best possible marketing plan.

c) Equipment

Optimization models are in use to determine car fleet sizing and investigate distribution strategies, e.g. reloading empty return movements and making alternate pool assignments. Intermodal studies are looking at increasing the trailer hitch utilization of the car fleet. Recent applications include the real-time forecasting of empty car availability by location.

d) Other Departments

Operations Research work in other departments covers the whole spectrum of railroading.

Major projects include:

- workload analysis and planning for Engineering;
- inventory control and stock ordering for Purchasing and Materials Management;
- hardware capacity planning simulations for data processing centres;
- microcomputer applications for budgeting and forecasting for financial managers;
- computer-aided simulations to analyze and design intermodal yards and domestic container systems.
- workload planning and scheduling for Mechanical Shops.

SUMMARY

Operations Research is a relatively new discipline within the railroad industry. O.R. groups are engaged in problem-solving and developing decision support systems for management in both the planning and real-time operations environments. The techniques of Operations Research have been successfully applied in almost every facet of railroading and are expected to play an increasingly important role.

REFERENCES

1. "Introduction to Operations Research", F. S. Hillier and G. J. Lieberman, Holden-Day Inc., 1980.
2. "Principles of Operations Research", H. M. Wagner, Prentice-Hall Inc., 1969.
3. "Proceedings: 1981 Fall Industrial Engineering Conference", Institute of Industrial Engineers, Industrial Engineering and Management Press, 1981.

COMMITTEE 22—ECONOMICS OF RAILWAY CONSTRUCTION AND MAINTENANCE

Report of Subcommittee No. 1

Use of Switch and Panel Exchanger Decoursey Yard, Kentucky, Seaboard System RR

B. G. Hudson (Subcommittee Chairman); R. W. Bailey; R. J. Begier; T. M. Brueske; W. J. English; C. H. Gaut; E. Q. Johnson; T. D. Mason; F. S. Mitchell; M. S. Reid; W. J. Semioli; E. H. Steel; J. T. Sullivan; J. D. Vaughn; J. T. Ward; B. J. Worley

The open site on the Seaboard System Railroad, just beyond a turnout at Decoursey, KY, where AREA's Committee 22 members gathered for a special demonstration of the Swedish Rail System *S*witch and *P*anel *E*xchanging System *(SPE)*, was not the type of location where the full benefits from use of such equipment can be fully perceived.

Rather, according to A.C. Parker, Jr., Seaboard's Chief Engineer-M/E Field Operations, the Seaboard System has found the equipment to be more helpful at locations where side or overhead clearances are severely limited. The actual track site chosen, however, was more amenable for the viewing of the operation by Committee onlookers.

Seaboard's SPE is being employed on switches and straight track for the installation of new track panels, and where the railroad wants to lower the track more than 6 inches. The track-contained type of operation afforded by the SPE, for example, in tunnels and on ballast deck bridges, minimizes clearance requirements.

The equipment operates under these tight conditions in an on-track mode and within the confines of ballast shoulder edges. Also, track panel raises are limited to about three feet. In locations where there are extremely restricted clearances — single track tunnels for example — the logistics for the preparation of ballast and/or subgrade, as well as spoil removal, has to be planned carefully in advance if grading operations are to be efficient. Access and large scale movements by grading equipment, and the final removal of spoil, among other tasks, are constrained in the direction of the tunnel track. As such, they could interfere unnecessarily with other track chores.

The equipment being demonstrated was developed in conjunction with the Swedish Rail System in the late 1960's for work on that country's track and was first introduced to railways outside of Sweden in 1977. Since then, the equipment has performed on timber and concrete tie track in Australia, North America, as well as other parts of Europe. Besides the Seaboard, the SPE's services in the U.S. includes yard switch replacement on the Richmond, Potomac and Frederickburg and on AMTRAK switch and straight track.

The SPE, however, is not the only system of its type that has seen service on American railroads. AMTRAK has also been using similar equipment of different manufacture, replacing switches along the Northeast Corridor in a mode of operation like that of the SPE.

During the demonstration on the Seaboard, a continuous 354-foot track panel was removed by the SPE; foul ballast graded out and the same panel subsequently returned and lowered into place — over newly placed geotextile for most of its length. This procedure conformed to the three basic steps in any SPE panel exchange cycle:

1. Removal of old turnout or track panel.

2. Preparation of the track bed.

3. Return and positioning of a new or the same panel.

The force used by the Seaboard at Decoursey was essentially a small yard gang and its equipment. Machinery included a tracked dozer, a multipurpose rubber-tired front-end loader with a 4-in-1 segmented bucket, and a medium-sized yard truck with high rail capability. In turn

the SPE's component units consisted of six sets each of power units, jacking beams, jacks, trollies and light track carts, and segments of temporary track.

After first cutting the 132-lb. (CWR) track at two points, the Seaboard yardmen spaced the SPE's jacking units along the then isolated track panel and raised it about three feet above the ballast—as part of the Step 1 procedure in the cycle. Temporary rail was then pulled in along the center line of the track and positioned under the elevated track superstructure. A consist of six trollies, each of 17,500-lb. rated capacity and linked by the tow bars, was next pushed from the running rail to the temporary rail by the medium hi-rail truck. Following this, the panel was lowered on the trollies, which were then pulled away along the running track by the rubber-tired loader, and thus exposing the ballast.

In Step 2, the loader removed the temporary rail and both the loader and the dozer graded out the old, foul ballast. In Step 3, which is essentially a reversal of Step 1, the temporary rail was repositioned using the loader and the trollies carrying the original panel brought back.

After removal of the temporary rail, the geotextile material was rolled underneath the still-elevated track panel. Burlap bags filled with ballast were then distributed on top of the geotextile material and underneath a tie in the panel. The jacks were used again to lower the panel into position onto the ballast bags which provided protection from damage to the geotextile while the panel was being aligned and reconnected. After the rail was connected, ballast was unloaded and the track was tamped.

Included at the rear of this report are elevation drawings describing the various stages of the SPE's work procedure. There is also a sequence of photos illustrating the Seaboard's operation, as it was viewed by Committee members.

Chief Engineer Parker explained that prior to the work being observed, track crews had double-spiked the ties to prevent their falling down during the panel lifting stage. He also stated that the removal of about 780 feet of panelized track was the Seaboard's best production rate during a one-day shift. The work was performed in a work block starting at 8:00 a.m. and closing at 2:00 p.m.

The capability of the equipment to negotiate curves depends, in part, both on the design of the trollies and on the track geometry encountered. Attached is a guideline for setting the required distances between the SPE trollies, as a function of curvature for a single curve.

Table 1 illustrates the allocation of SPE trollies, power packs and lifting units for various turnout types and track panels (timber tie track). From prior experience, the manufacturer estimates that the normal working crew assigned to an SPE operation is one man per jacking unit, a supervising forman and an additional three to four men for handling the assisting equipment and temporary rail.

The SPE, as it is being used on the Seaboard, has been modified slightly from standard to fit specific requirements by the railroad. Because of the lengthy panel work for which the equipment is being used, long telescopic bars have replaced the standard types. These, combined with the six sets of lifting units and trollies, has given the equipment a rated panel length capability up to 390 feet. Enough temporary track panels have been supplied to the Seaboard—about 420 feet in 13-foot segments—to handle extra-long panels. In fact, during a previous operation, the Seaboard was able to remove a 424-foot panel.

Normally, small gasoline engines are used to power the lifting jacks. In Seaboard's case, however, these were replaced by 5-HP diesel units. The diesels do not emit the high concentrations of carbon monoxide associated with the use of gasoline engines—a very important safety feature for tunnel work. Another safety feature required by the Seaboard has been the set of six light track carts, observed in use during the demonstration. Jack operators could activate their equipment while standing on the ties. But as an extra safety measure, the track carts serve as work platforms, and also as carriers for the power units and jacking equipment.

Besides general utility in confined locations, a number of other benefits are cited for the use of this equipment. They include:

1. Avoidance of damage in removing or replacing track panels. The panels can be moved easily on track to a lead-off serving an on-site dismantling or reclamation facility.
2. The trollies can be used to carry panels substantial distances—at travel speeds up to 24 MPH.
3. During removal and installation of the panels, a crane is not required. However, a crane might be needed at the panel building and/or off-loading area to set or remove the panels from SPE trollies.
4. The number of power, trolley and jacking sets can be varied to fit the lengths of panels being handled.
5. Among other devices, each trolley is fitted with a lateral movement device for shifting a panel as much as 17 inches laterally, on either side, during transportation. This feature assists in keeping loads balanced when sharp curves are being negotiated during travel.
6. Light-traction vehicles, such as an on-track crane or ballast regulator, can provide motive power to the trolley system and its loads.
7. A normal set of six jacking units can be transported either off the road by truck or on an ordinary flat car.
8. There is no fouling of adjacent track, and minimal cleanup is required at the track site.
9. Parts interchangeability and ease of replacement of lifting cylinders, jacking beams, power and hydraulic units and trollies are said to reduce equipment downtime and to minimize job interruption.
10. Along with standard lifting beams, "lateral movement lifting beams", also hydraulic, can be supplied to permit workmen to "walk" panels laterally to and from the track at alongside locations where clearances permit. This can eliminate the need for trollies or cranes, for example, when turnouts can be built adjacent to their installation locations.
11. Operator and labor tasks are easily learned, and do not require a high degree of prior skill.

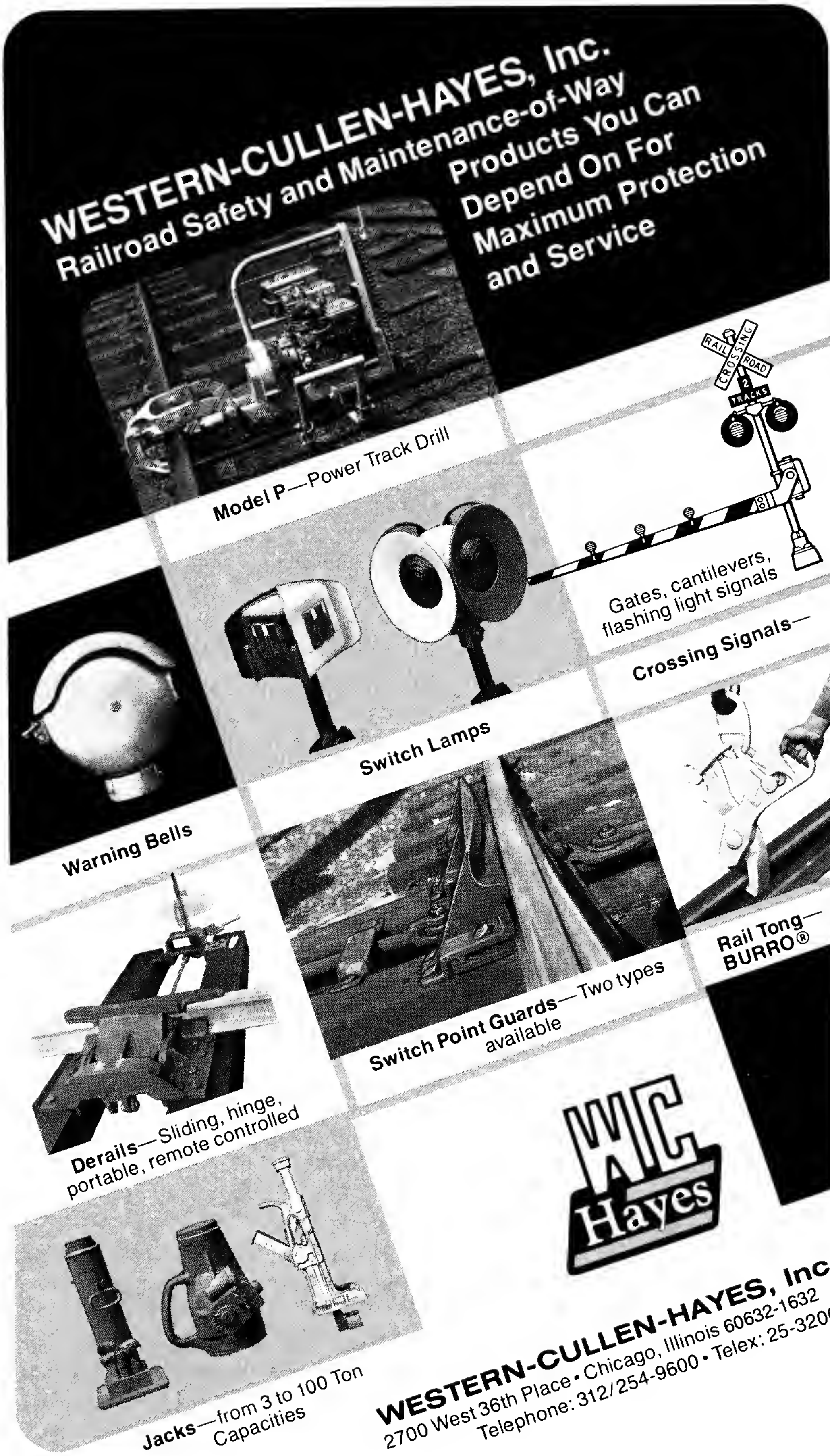
CONCLUSION

As best as can be determined, neither the Seaboard System Railroad or other American roads, which have been using the SPE, have specific cost breakdowns which compare its economics against other methods for similar track rehabilitation tasks.

Economic judgements vary from case to case, but tend to be relatively general, being associated with particular site conditions and needs, for which the flexibility and facility gained with the SPE's specific mode of operation are helpful.

Besides the benefit of working in tight locations, as already has been noted, the equipment also provides efficiency in the deep undercutting of long turnouts. It cuts beyond 6-8 inches, the full removal of the track panel permits unrestricted access to underlying foul ballast which leads to faster rehabilitation than could be obtained by multiple undercuts.

Also, it was explained that during the replacement or return of panels, necessary track adjustments can be made more easily with the SPE method. At the same time, the Seaboard agrees that less damage to the panel structure and its components occurs during the handling process.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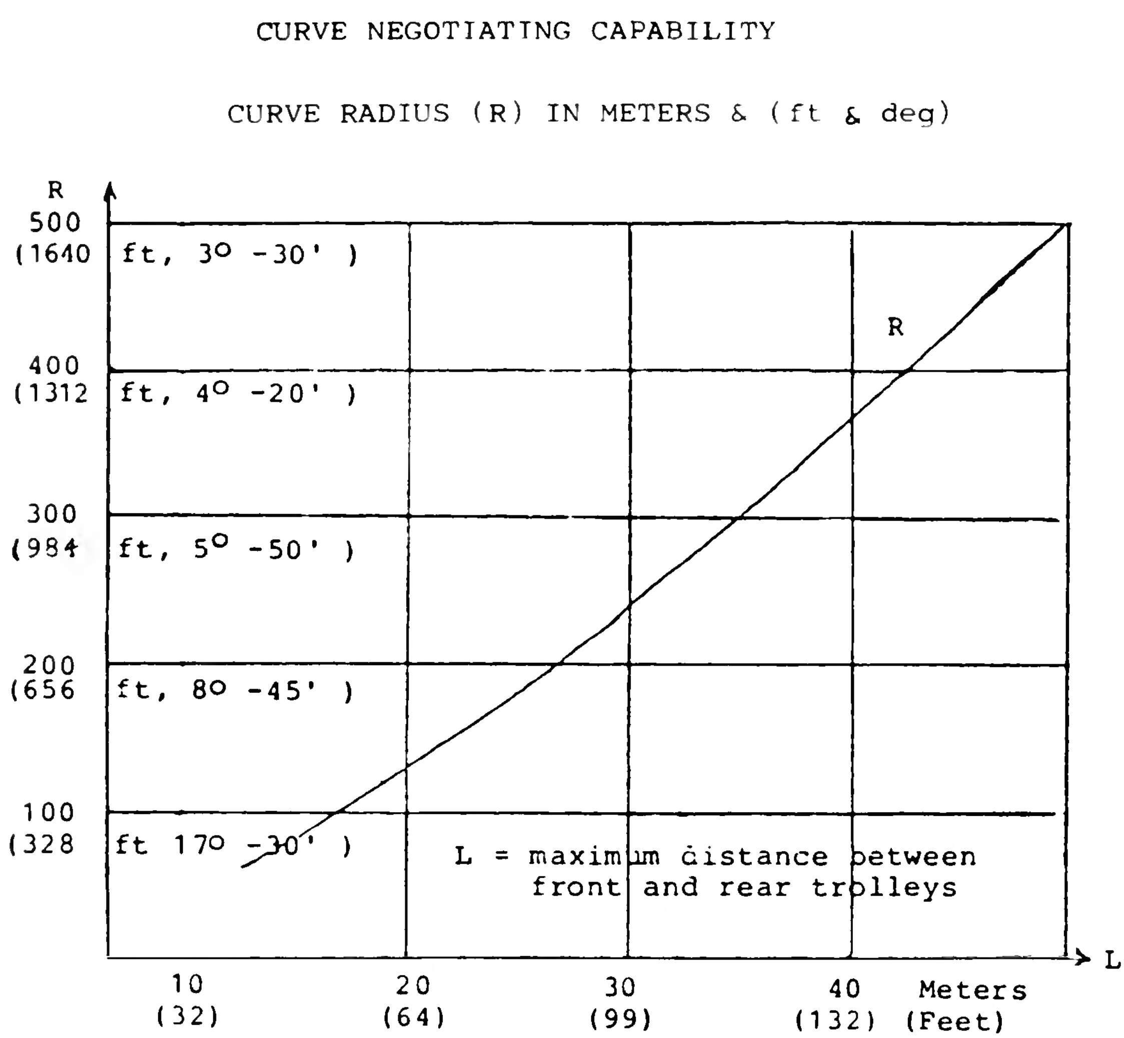
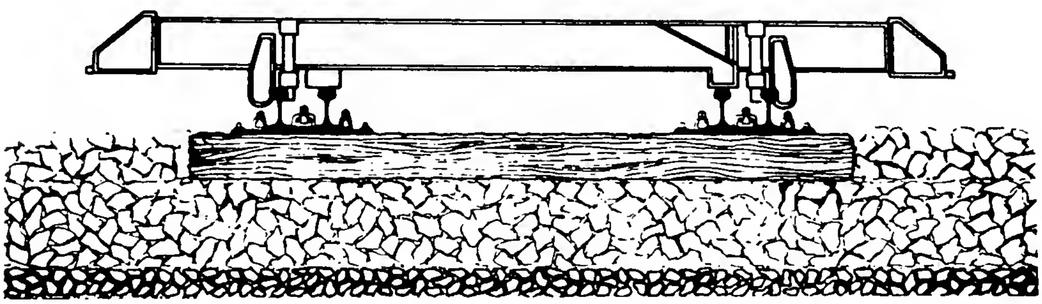


FIG. 1 Track Curvature (Single Curve) versus Trolley Spacing.

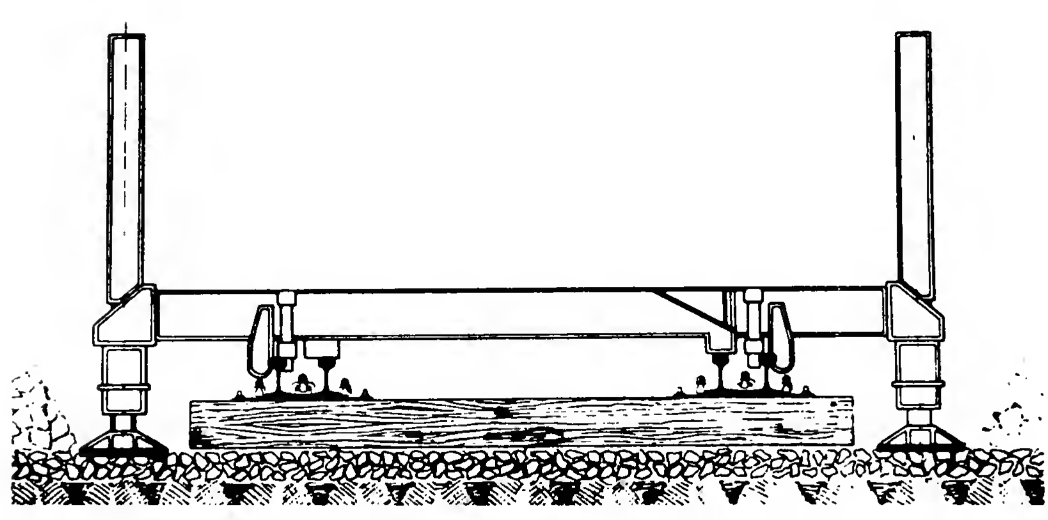
Table 1. SPE Units per Panel Type.

Type of track section	Number of units required
No. 10 turnout	3
No. 15 turnout	4
No. 20 turnout	5
No. 20 crossover section	6 running on double track
Prefabricated panels	1 unit per each 12 meter (68 feet).

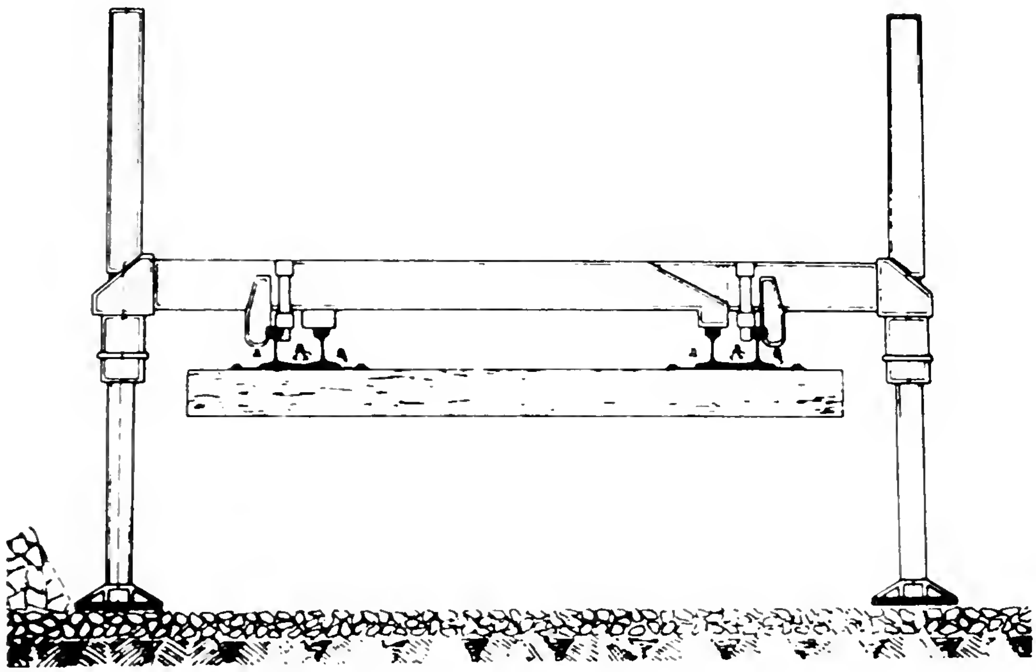
Illustration of Working Procedure for SPE.



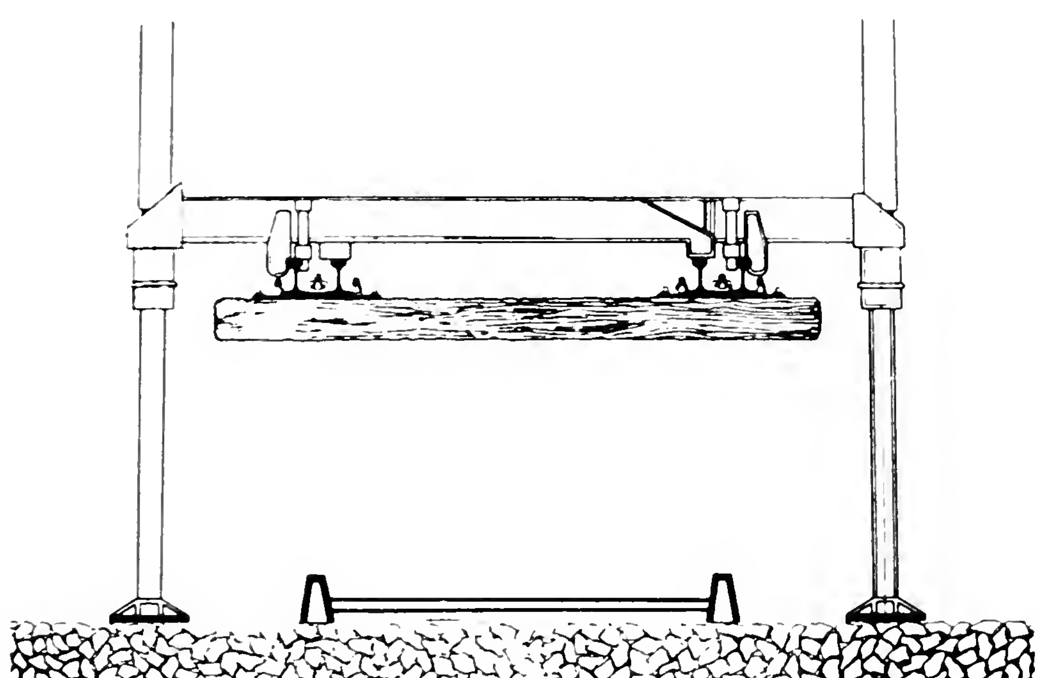
Mounting of lifting beams



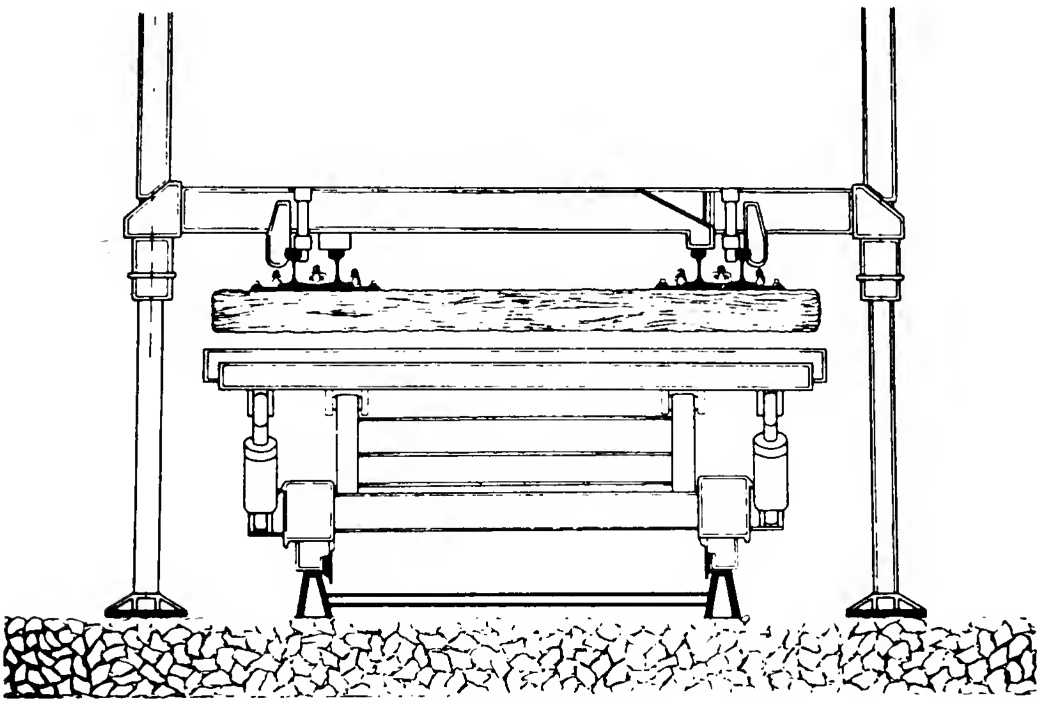
Mounting of lifting cylinders, placing of base plates. Removal of ballast not necessary.



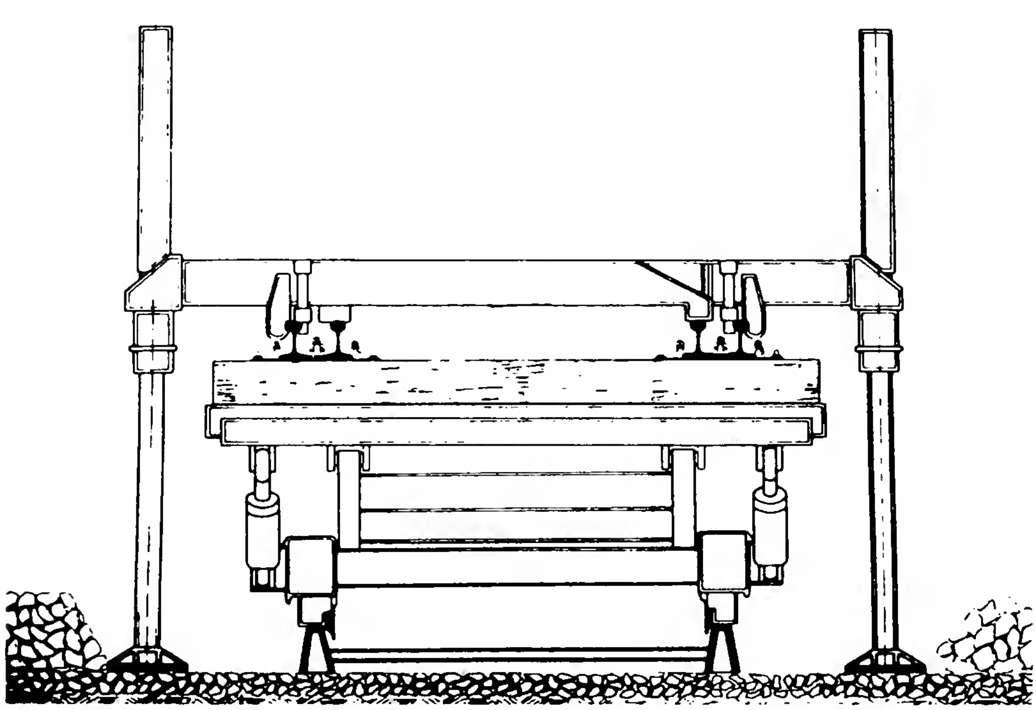
Hoisting of the panel.



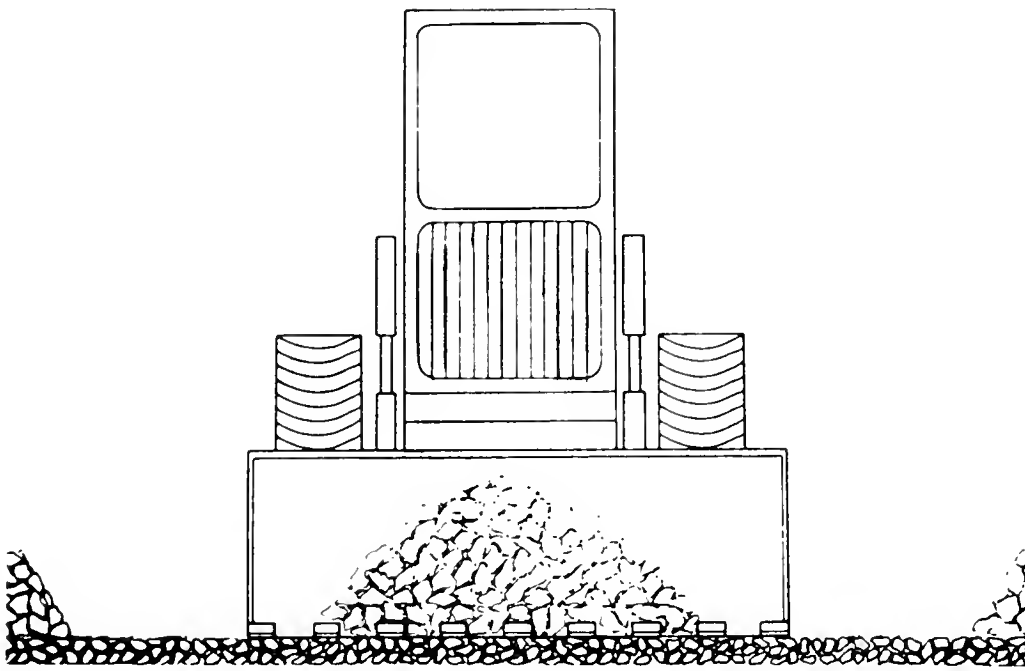
Temporary track pulled into position.



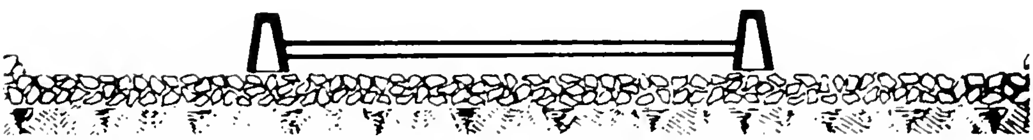
Rolling the trolleys into position.



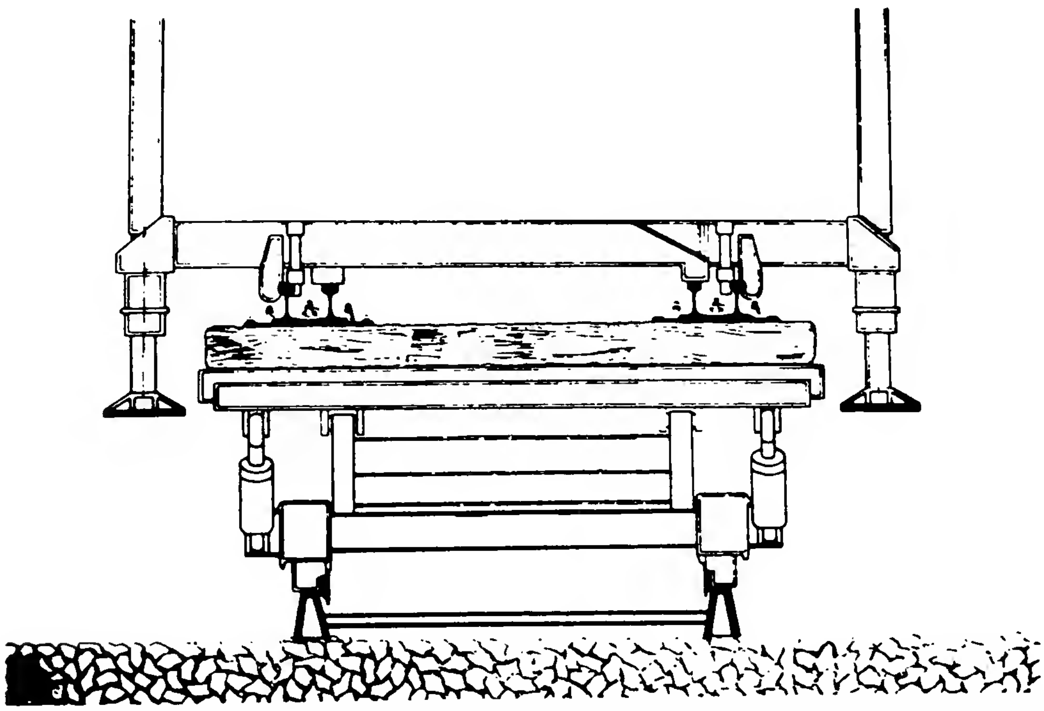
Placing the panel on the trolleys.



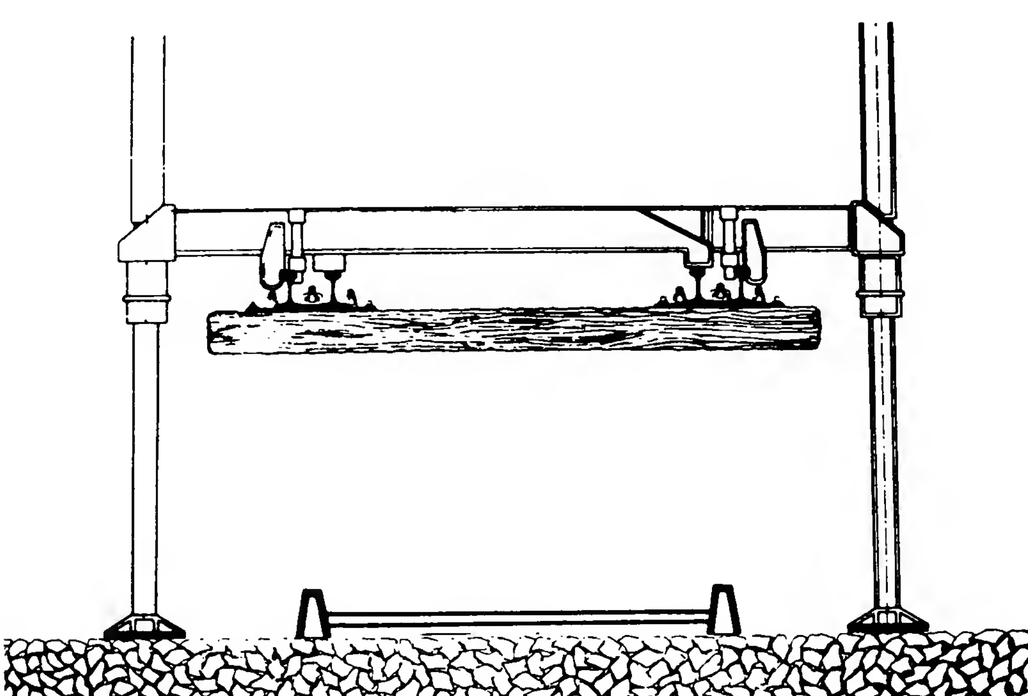
Rehabilitation of the ballast bed.



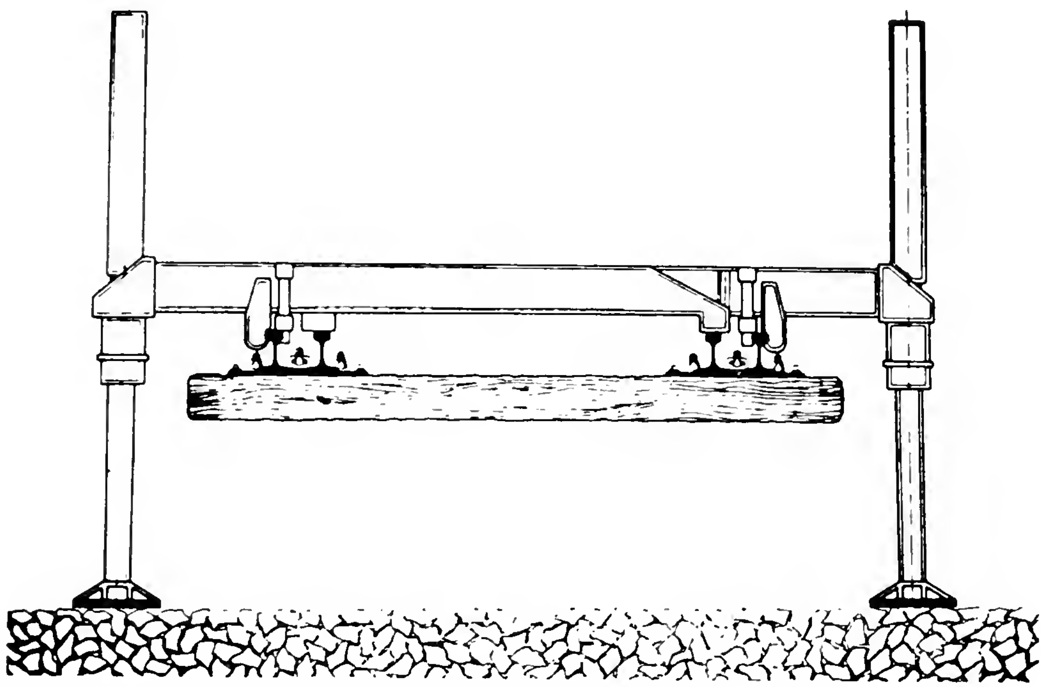
Temporary rail pulled back into position.



Arriving with the new panel. Final adjustment of the position when the panel is resting on the trolleys.



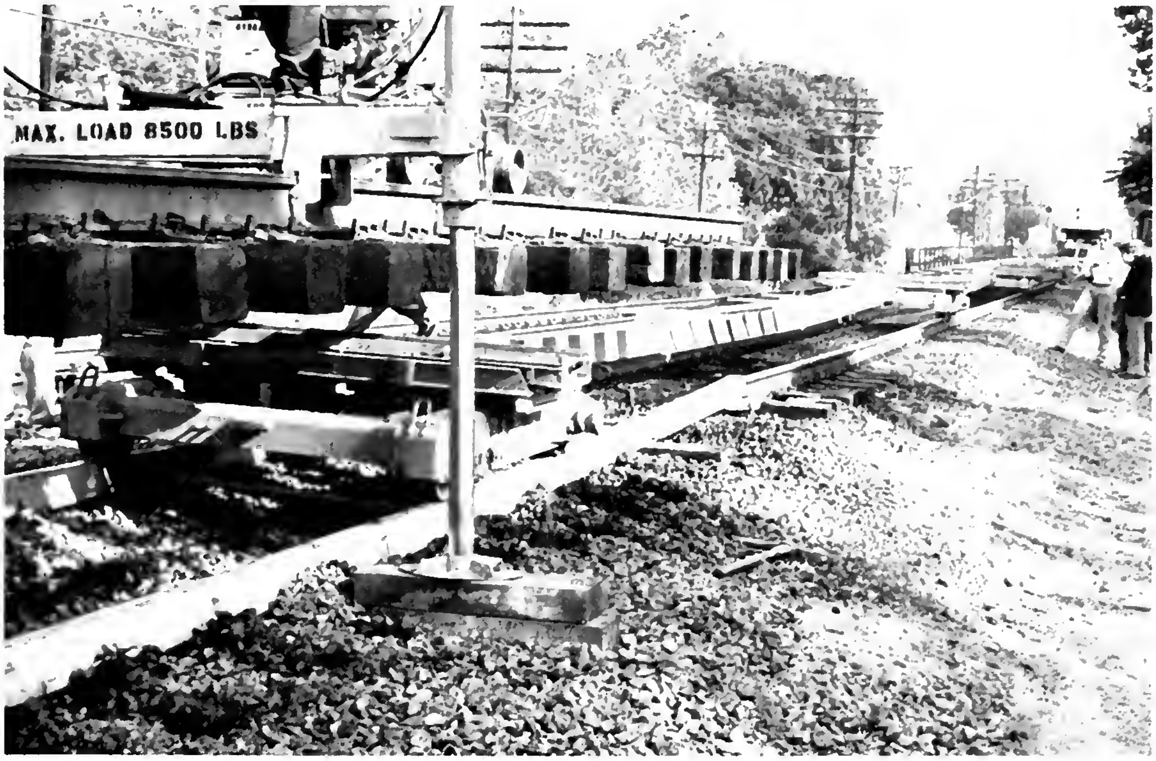
Removing trolleys.



Lowering the panel.

New panel in position—ready for ballast work.

As railroad personnel and distant AREA Committee 22 members look on, six separate jacking units elevate a 354-foot long track panel.



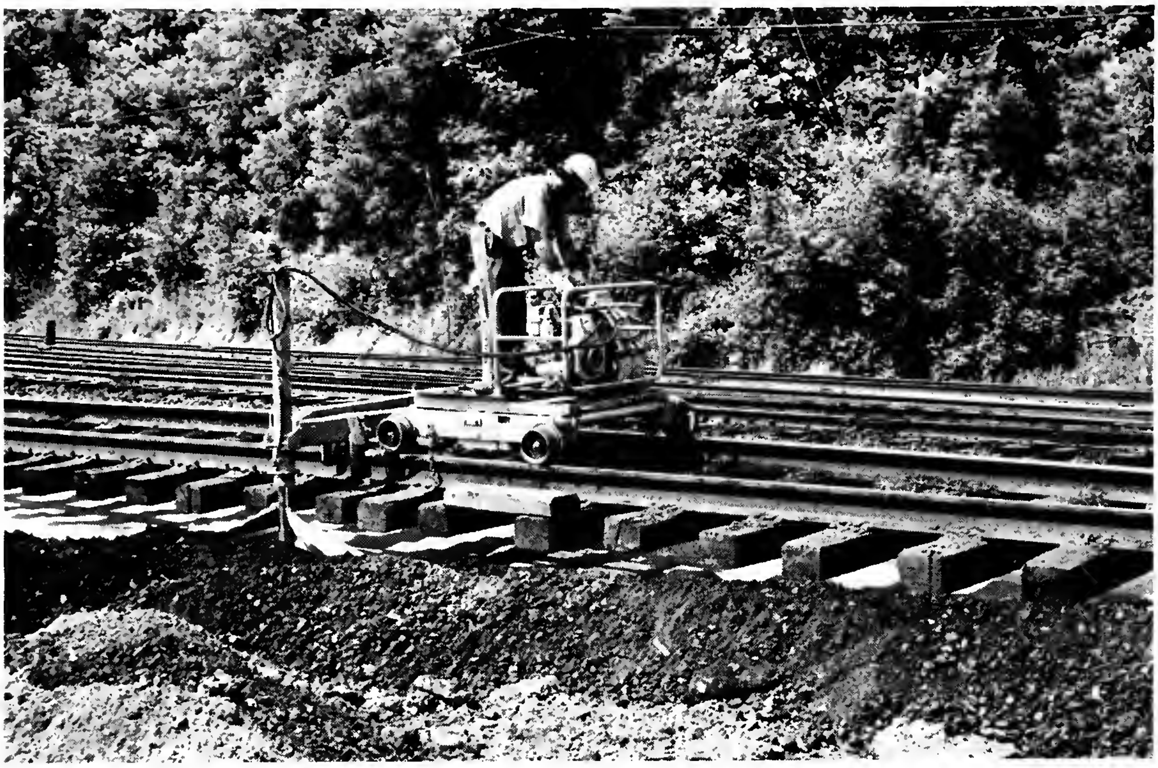
Once the panel is elevated, temporary track is installed atop the fouled ballast and panel trollies moved underneath.

After the panel is lowered into the trolley consist, it is then towed out of the way along existing track.

With the track cleared, the fouled ballast material is quickly plowed off to one side for later pick-up.

In addition to a tracked dozer, the Seaboard used a loader with a segmented bucket to assist in grading.

Upon completion of the grading, temporary track is again installed and the same track panel is returned.



On one of the six track carts used to carry the power units and jacking controls, operator gauges panel descent.

MEMOIRS

Charles Rayford Fulghum
1931-1983

Charles Rayford (Ray) Fulghum, System Engineer-Production and Materials of the Illinois Central Gulf Railroad, died on September 11, 1983 at South Suburban Hospital in Hazel Crest, Illinois.

Ray was born on April 20, 1931 at Jackson, Tennessee. He attended Marion Military Inst., Marion, Alabama and graduated from Auburn University, Auburn, Alabama with a B. S. Degree in Civil Engineering in 1954. He was married to Jane Mann on November 27, 1958 at Springfield, Illinois.

In April of 1954 Mr. Fulghum began his railroad career with the former Illinois Central Railroad as an Instrument Man on the Chicago Terminal. All but for the two years between June 1954 and June of 1956, when he served his country as a member of the USAF, Ray spent with the Illinois Central and the Illinois Central Gulf in various positions. He was Supervisor Trains and Track, switched to Operating Department in June of 1969 as a Trainmaster and back to Engineering Department in January 1970 as Assistant Division Engineer at Champaign, Illinois and to Division Engineer at Carbondale, Illinois in March of 1972. In June of 1975 he returned to Chicago in the Vice President & Chief Engineers Office as System Engineer Planning then Maintenance Operation Engineer, System Engineer-Rehabilitation and in June of 1979 till his death as System Engineer-Production and Materials.

Mr. Fulghum became a member of the American Railway Engineering Association in 1959. He was appointed to Committee 5, Track in February of 1977 and was on Subcommittees 4 and 6 at the time of his death.

Surviving Ray are his wife Jane and a son David, both of Glenwood, Illinois.

All of his associates and friends are saddened to hear of his passing.

S. W. Brunner

Frank R. Woolford
1901-1983

Frank R. Woolford, retired Chief Engineer of the Western Pacific died August 15, 1983 at Marin General Hospital outside of San Francisco. He is survived by his wife, Kathleen. Mr. Woolford was born in Little Rock, Arkansas on August 14, 1901. He began his railroad career on the Missouri Pacific as a rodman in the 1920s and progressed to division engineer before becoming engineer-maintenance of way and structures on the Western Pacific in 1949. In late 1949, he was promoted to chief engineer of the Western Pacific, a position he held until 1966 when he retired. After retiring from the railroad, Mr. Woolford established his own railroad consulting firm and was active until his death.

Mr. Woolford joined the AREA in 1927 and served on Committee 22 from 1952 through 1981. He was elected Member Emeritus in 1968. He served as director of the AREA from 1955 to 1956 and as its president from 1959 to 1960. Committee 22 expresses its sympathy and sorrow in his death.

A. E. Shaw, Jr.

O'NEILL & GASPARDO

CERTIFIED PUBLIC ACCOUNTANTS

837 EAST 162ND STREET • P.O. BOX 83
SOUTH HOLLAND, ILLINOIS 60473
333-4000

January 24, 1984

ACCOUNTANTS' REPORT

Board of Directors
AMERICAN RAILWAY ENGINEERING ASSOCIATION
2000 L Street N.W.
Washington, D.C. 20036

We have examined the balance sheet of AMERICAN RAILWAY ENGINEERING ASSOCIATION as of December 31, 1983 and the related statement of income and expenses for the year then ended. Our examination was made in accordance with generally accepted auditing standards and accordingly included such tests of the accounting records and other auditing procedures as we considered necessary in the circumstances.

In our opinion, the accompanying balance sheet as of December 31, 1983 fairly presents the financial condition of AMERICAN RAILWAY ENGINEERING ASSOCIATION in conformity with generally accepted accounting principles applied on a consistent basis.

O'Neill + Gaspardo

O'NEILL & GASPARDO

American Railway Engineering Association
Balance Sheet
December 31, 1983

ASSETS

Current Assets		
Petty Cash	$ 50.00	
National Bank of Washington-Checking	5,519.66	
National Bank of Washington-Savings	14,716.41	
American Savings & Loan Association - Stockton, Cal., Certificate of Deposit due April 4, 1984	100,000.00	
Community Savings & Loan - Now Account	114,328.27	
Investment - Seligman Mutual Fund	12,842.25	
Accounts Receivable - Dues	1,330.00	
Inventories	59,384.50	
Accrued Interest Receivable	861.11	
Total Current Assets		$ 309,032.20
Fixed Assets		
Furniture & Fixtures	$ 6,143.71	
Less: Accumulated Depreciation	2,633.01	
		3,510.70
TOTAL ASSETS		$ 312,542.90

LIABILITIES AND MEMBERS' EQUITY

Current Liabilities		
Expenses Payable		$ 2,733.65
MEMBERS' EQUITY		
Balance, January 1, 1983	$ 172,572.29	
Plus: Excess of Income over Expenses year ended December 31, 1983	137,236.96	
Total Members' Equity		309,809.25
TOTAL LIABILITIES AND MEMBERS' EQUITY		$ 312,542.90

American Railway Engineering Association
Comparative Statement of Income and Expenses
Years Ending December 31,

	1983	1982
INCOME		
Dues	$ 112,558.57	$ 98,399.07
Publications -		
Bulletins	11,678.50	12,066.60
Proceedings	1,818.00	2,360.00
Manual	157,316.40	121,772.34
Portfolio	35,098.77	39,619.62
Miscellaneous	2,108.95	2,180.93
Advertising	34,164.51	28,507.01
Interest	17,180.26	19,213.09
Conferences	21,547.00	11,865.70
Miscellaneous Income	10,533.09	9,797.05
TOTAL INCOME	$ 404,004.05	$ 345,781.41
EXPENSES		
Cost of Publications		
Inventory January 1, 1983	$ 70,193.00	$ 63,560.00
Manuals	20,563.93	59,302.64
Portfolios	5,083.23	16,837.44
Bulletins and Proceedings	35,600.59	76,360.48
	131,440.75	216,060.56
Less: Inventory December 31, 1983	59,384.50	70,193.00
Net Cost of Publications	72,056.25	145,867.56
Salaries, Taxes and Fringe Benefits	100,421.53	122,620.31
Stationery and Printing	5,548.93	14,532.50
Shipping and Phone	19,320.88	19,266.19
Supplies	1,813.63	2,529.53
Professor Expenses	3,932.06	16,029.72
Travel Expense	8,931.40	7,950.46
Conferences and Meetings	10,348.09	17,976.93
Audit	2,332.03	1,983.63
Temporary Help	2,462.61	952.63
Advertising	14,977.18	14,555.10
Student Design Competition Awards	2,827.95	-0-
Subscriptions AREA Members	10,485.13	10,429.77
Dues and Subscriptions	1,410.00	1,082.00
Miscellaneous Office Supplies and Refunds	7,126.55	13,854.17
Insurance	1,895.20	-0-
Depreciation	877.67	877.67
	266,767.09	390,508.17
EXCESS OF INCOME OVER EXPENSES OR (DEFICIT)	$ 137,236.96	$ (44,726.76)

American Railway Engineering Association
Notes to Financial Statements
December 31, 1983

NOTE 1. SIGNIFICANT ACCOUNTING POLICIES

Accounting Method - The association's books and records are maintained on the cash basis. The enclosed statements were prepared from the books after adjusting to the accrual basis.

Fixed Assets - Depreciation of furniture and fixtures is provided using the straight line method over the estimated useful life of each asset. Individual assets purchased for less than $500.00 are not capitalized.

Inventories - Inventories are stated at the lower of cost or market using the first in - first out method.

NOTE 2. FEDERAL INCOME TAX EXEMPTION

The association is exempt from federal income taxes under Internal Revenue Code Section #501(c) (6). Federal Form 990 Return of Organization Exempt from Income Tax has been prepared in conformity with this report.

DIRECTORY OF CONSULTING ENGINEERS

NOTES

American Railway Engineering Association

Bulletin 697

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CONTENTS

Cover Photo: Eastbound unit coal train on Burlington Northern's Lincoln-Alliance, Nebraska Line west of Ravenna, Nebraska July 3, 1984.

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The Busiest Rails on Earth?

The Burlington Northern's line from Alliance to Lincoln, Nebraska, carries the highest annual line-haul tonnage over any single track line in the United States, and probably the world. Its tonnage, consisting mostly of unit trains of coal from Wyoming (see cover), has approached 120 Million Gross Tons (M.G.T.) per year and may exceed this in 1984. The oldest of the all-welded rail on this line consists of A.R.E.A. 132 lb. section rolled in 1974. This rail has experienced approximately 650 million gross tons of use. Rail head profile grinding may extend its life considerably beyond this.

20 years ago the line was a secondary main line with perhaps 7-8 M.G.T. per year. It was upgraded as coal traffic began and expanded dramatically, but ties were wisely not renewed out-of-face since many 8-ft. ties are still performing well even after more than 40 years service, even 45 years in some cases. (Note the shorter ties in the photograph above). The line has frequent double track sections with crossovers to increase line capacity and add operational flexibility, and additional second track is being added (see photo below) in certain locations.

The photo above shows Burlington Northern's Lincoln-Alliance line east of Hyannis, Nebraska.

The photo at left shows piling ready for second bridge to allow construction of second track west of Ravenna, Nebraska.

Both photos taken July 3, 1984.

Presidential Address

Hubert L. Rose*

At this point in the program it is the traditional duty and privilege of your president to highlight the activities and accomplishments of the past year.

I'd like to express a special welcome to the ladies who are here for the opening session and to welcome you to attend any of the sessions which may be of interest to you. I hope you will join us Wednesday morning when our new officers are installed.

The past year has been a very productive year for our association. I am pleased to advise that our membership as of February 22, 1984, stood at an all-time high of 4,180 members. This is an increase of 175 members over the past year and is a good indicator of a healthy organization. It is vital to our organization that we promote the professional growth of our membership. This association is a professional organization in the private sector that can effectively deal with the overall engineering problems and challenges of the railroad industry.

Our association is in good financial condition. This past year has been one of our best financial years. While belt-tightening has gone on this past year on most railroad properties—our finance committee and headquarters staff have also done an excellent job in controlling expenses of the association. This, along with increased revenue from sale of AREA publications, has produced a good financial year.

One of the highlights of this past year has been the Regional Meeting. The concept of the regional meetings was conceived in 1965 to bring technical reports or presentations to our members in the geographic segments of the North American continent who do not have the opportunity to attend the annual technical conference.

The 1983 Regional Conference was held last fall in Danvers, Mass., under the enthusiastic direction of Mr. V.R. Terrill, our senior first vice president, who did an outstanding job of presenting a very informative program. This technical program was beneficial to those 255 engineering professionals and guests in attendance.

A highlight of the year 1984 will be the regional meeting to be held in Mexico City October 25 under the direction of Mr. Francisco Estopier. An outstanding feature of this regional meeting will be an inspection trip plus a field trip to a concrete tie plant. I hope that many of our members will attend this session as traveling expenses to Mexico City will be no more than to many towns in this country.

In the year 1967 the AREA constitution was changed to require geographical distribution of membership on the Board of Direction and of elected members of the Nominating Committee. The 1968 elections were held on the new regional basis. However, with the many mergers of railroads in recent years the regional concept has become outdated.

During the year 1983, over 98 percent of the membership approved a change in Article VI of our constitution. The basic change was from the regional concept, which required a designated number of directors from the East, West, and South over a 3 year period to—an at-large concept in this country requiring 9 directors to be elected at-large from the United States with two additional board members from Canada and one from Mexico. This constitutional change will provide our association the power to maintain territorial balance as well as to elect the most competent members to the Board of Direction.

Another constitution change as approved by the members in 1983 was the streamlining of the membership election and reinstatement process which eliminated unnecessary time and paper work from the process.

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*Assistant Vice President-MW&S, Southern Railway System

The purpose of this association as stated in the constitution is as follows: "The object of the Association shall be the advancement of knowledge pertaining to the scientific and economic location, construction, and maintenance of railways."

This "advancement of knowledge" is the goal of our 21 Technical Committees who are really the heart and soul of our organization.

Each committee is charged with the dual responsibility of first, developing facts and information pertinent to their assigned technical subjects for presentation to the membership as information in the four bulletins published each year; and second, the formulation of recommended practices to be submitted for adoption and publication in the Manual for Railway Engineering or Portfolio of Trackwork Plans.

The activities of the AREA technical committees continue at a high level. In 1983, a greater volume of changes and new material went into the manual than any recent year. These revisions were published and made effective August 1, 1983.

In 1984 many changes are being considered and will be made. Committee 1 has developed technical information on flood control determination and culvert design for manual consideration.

Committee 3 is revising Chapter 3 on crossties and has completed Part 1 - Timber Crossties while working closely with the Railway Tie Association and the American Wood-Preserver's Association towards the development of a common timber crosstie specification including a new classification for crossties.

Committee 4 has updated the Specifications for Steel Rails to include the current steel manufacturing processes and requirements for a better rail.

Committee 5 has submitted material for consideration regarding industrial track and tamping as well as a specification for rail anchors.

Committee 6 has updated the specifications and design criteria for railway buildings.

Committee 8 has submitted a new specification on drilled shaft foundations as an addition to Chapter 8 for the manual.

Committee 10 - Concrete Ties and Committee 13 - Environmental Engineering have submitted updated technical material for manual consideration.

Committee 15 after working two years to get an entirely updated Chapter 15 on steel bridges into the manual in 1983 has now submitted extremely useful guidelines for evaluating fire-damaged steel railway bridges.

Committee 28 has submitted updated technical information on clearances and Committee 33 has requested that a section on catenary design for electrified railroads be added to the manual.

These are only part of the activities of the 21 technical committees. Many committees have submitted bulletin information over the past year. The association is thankful to you for the long hours you have put into your assignments.

It might be mentioned that many technical subjects that have been studied in the past need to be updated with changing conditions. As an example—Committee 16 after extensive tests on a 45-mile stretch of track of 6-degree curves on the old Denver and Salt Lake Railway with 14 wayside flange lubricators in 1939 made the following statement: "Flange lubricators properly located and properly functioning reduced curve resistance approximately 50 percent."

With heavier wheel loads today, different rail metallurgy, various lubrication systems and types of greases—lubrication is an important subject for research to obtain maximum benefits on rail wear, wheel wear and energy savings. Thus, it is again an assigned subject for Committee 4 to study and is also being given extensive research at FAST by AAR and on several railroads.

I would like to say to all committees that a date should be set for completion of each assigned subject. Reports must be factual with updated technical information but they should be timely or else committee members lose interest in their work and the association loses the benefit of needed information.

AREA has continued to work with the Transportation Systems Center at Cambridge, Mass., in the development of track performance standards. Several field tests have been run on the Boston and Maine, Santa Fe, Chessie, Southern, and FAST towards getting factual data on rail restraint, track geometry for cross level, twist, and alignment, track stability and buckling concepts, rail defects and rail integrity. With the current AREA track performance committee under the able chairmanship of Ron Drucker, Chief Engineer of Chessie and the committee working with the capable engineers at TSC, it is felt that the comprehensive and analytical approach of developing track performance standards can be a meaningful system for determining track safety.

REMSA plays an important part in the well-being of our railroad industry. Through their research and development, they are constantly updating and improving the materials, machines and service which we need to improve our plant and our productivity. With the cost of a laborer today approaching $35,000 per year (including fringe benefits) it is imperative that engineers reduce costly manpower through improved mechanization in construction and maintenance operations. With tight budgets it is more important than ever to increase productivity through improved machinery and our friends from REMSA are helping us do just that. We appreciate their efforts.

As this conference begins, I would like at this time to thank our Conference Operating Committee under the able leadership of Eric Peterson for their outstanding dedicated work which is so essential for a successful technical conference.

AREA headquarters staff under the excellent leadership of our Executive Director Lou Cerny have done an outstanding job for our association over the past year. The results of their performance are shown by this being a vibrant technical organization.

Through the continued efforts of our members, committees, and Board of Direction in the advancement of knowledge in location, construction, and maintenance of railways—we will maintain a professional association for the progress of North American railroading.

Thank you.

Empirical Rail Wear Model
AAR Report No. WP-104

I.A. Reiner* and D.E. Staplin**

EXECUTIVE SUMMARY

Rail wear and fatigue are the primary reasons for rail replacement, and the ability to predict rail wear is important for both economic and planning purposes.

This Rail Wear Model is an empirical tool for predicting standard carbon rail head wear from tonnage and wheel loads. Variations in annual tonnage density, track gradient and curvature are also taken into consideration. It is a modified version of one that was previously calibrated in Europe and later refined by the Chessie System.

The model's predictions for the wear of standard carbon rail in main line track compare favorably with published data, where the loading patterns were known. The model's general form is also believed to be valid for other rail chemistries, but relatively little rail wear data is available for calibration.

Model calculations can be made on either a pocket or programmable calculator. The use of a larger computer would be desirable, if traffic information were available on a site-specific basis.

1.0 INTRODUCTION

The overall interest in tools for predicting the behavior of various railway track components has increased in recent years. Those interested include track maintenance officers, faced with increased maintenance requirements under ever-increasing loads; marketing departments striving to improve commercial decisions under deregulation; and top management faced with a need for significant growth in return on investment. Improved decision-making tools could assist in meeting the concerns of management at all levels in the rail industry. Use of the Rail Wear Model described in this paper can be made for costing and engineering purposes.

The development of an adequate empirical approach to rail wear is important because:

1. No adequate theoretical description of rail wear exists;
2. Wear is a factor in other forms of rail degradation, such as fatigue;
3. Wear represents a major reason for rail replacement. Railroads are currently buying and installing nearly a million tons of new rail a year, at a cost exceeding one billion dollars.

This Rail Wear Model is adapted from European work which was introduced to the United States by the Chessie System in 1970. It is an empirical tool which predicts loss of rail head metal with the passage of tonnage and time. Variations in the wheel loads, track gradient and curvature are also considered. The model has been shown to give valid results for most conditions associated with main line trackage.

The form of the Rail Wear Model is simple enough so that it can be used on a programmable pocket calculator. Results for generalized cases can be derived in tabular form for easy reference. If many repetitive calculations are required, or if traffic data are available, it can be easily programmed on a large computer. Such an application is included in the Appendix, Section 6.

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* General Supervisor System Planning, Chessie System

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1.1 Rail Degradation

Rail is progressively worn from the action of numerous influences, such as:

1. **Characteristics of traffic:** traffic volume, operating speeds, wheel load distributions, axle spacings, wheel diameters and nature of the vehicles' suspension systems;

2. **Characteristics of the track:** horizontal alignment, profile, type and condition of the track structure, condition of the ballast, subgrade and drainage, and presence or absence of rail lubrication;

3. **Characteristics of the rail:** inclusions in the grain structure of the steel, hardness, impact resistance, cross-sectional area and design;

4. **Characteristics of the environment:** amount and frequency of precipitation, temperature extremes, humidity and corrosivity.

Although there are several modes of degradation, wear and fatigue account for a high percentage of rail removed from track.

Rail wear is the cross-sectional area loss caused by passing wheels and atmospheric conditions. As the wheels roll (and in some cases, slide laterally and longitudinally) on the rail head, material is removed from the contact areas in the form of small particles. Metal deformation caused by contact stresses at the contact point also assists in removing metal. A third cause of wear is corrosion from atmospheric conditions, including moisture and the presence of chemicals.

Typical wear patterns in tangent track include the removal of metal from the top of the head and the plastic flow of metal towards the field side, as shown in Figure 1. With the advent of heavier cars (100-tons and greater), evidence of plastic flow to the gage corner has also been noted. The loss of vertical head height does not present a problem in itself, however, passing wheel flanges may eventually strike the top surfaces of the joint bars in bolted rail territory, resulting in undesirable maintenance conditions. More importantly, as wear proceeds, certain stress levels in the rail increase, which may contribute to fatigue.

In curves, the contact of wheel flanges with the outside or high rail causes wear in the horizontal direction, as shown in Figure 2. As this wear progresses, gage widening results, which requires corrective action. The lateral loading on curves have also been shown to contribute to higher fatigue defect formation rates [1].*

In a few instances, such as at highway crossings and in tunnels, etc., corrosion may cause rapid rail deterioration from surface pitting.

Rail fatigue results from high, repetitive stresses due to bending, wheel contact pressures and inclusions in the metal's grain structure. Stress concentrations at fillets, bolt holes and corrosion pits may also result in fatigue cracks. Inclusion of the rail wear and fatigue models in a more sophisticated planning tool has been described elsewhere [2].

* Numbers in square brackets [ ] refer to the References, listed in Section 5.0 of this report.

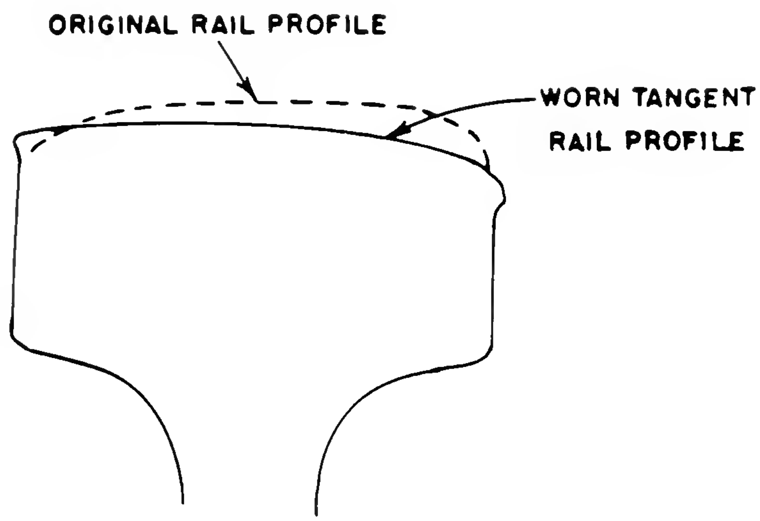

FIG. 1. Typical Tangent Track Rail Head Wear Pattern.

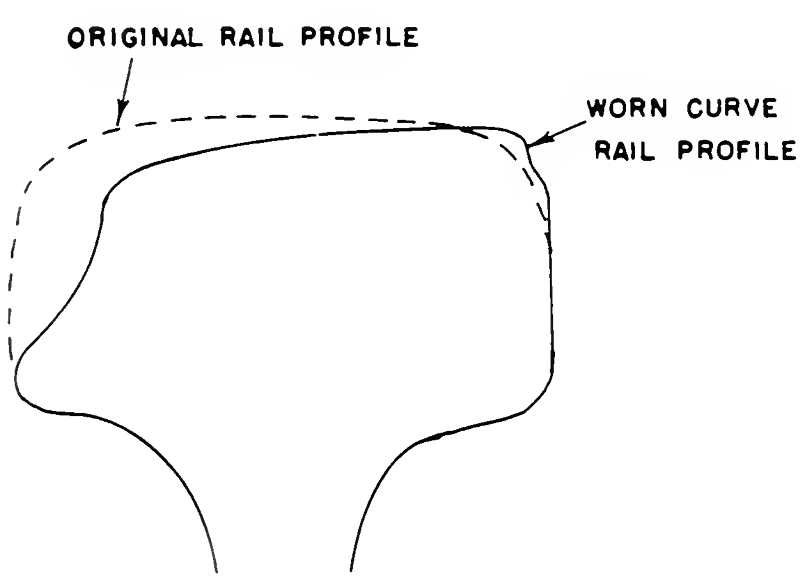

FIG. 2. Typical Curved Track Rail Head Wear Pattern.

2.0 DEVELOPMENT OF THE RAIL WEAR MODEL

2.1 The Couard-Gant Model

The present Rail Wear Model has as its basis the Couard-Gant Model, which expresses rail life in years as the quotient of allowable wear and the annual wear rate, as follows:

$$L_y = \frac{W}{T(a+bD^x)\,(1+dg^2) + c} \tag{1}$$

where:

L_y = life of the rail (years);

W = allowable wear (in.^{2});

T = annual traffic density (MGT/yr.); [MGT = million gross tons]

D = curvature (degrees);

g = gradient (percent);

a = wear rate of rail in tangent track (in.^{2}/MGT) + 0.0007;

b = wear rate of the high rail in curved track (in.^{2}/MGT/(degree)^x) = 0.000173;

c = annual rate of corrosion, equal to 0.0045 (in.^{2}/yr.) for tangent track, and 0.0052 for the high rail in curved track;

d = a constant relating the wear on grades to that on level track = 0.023;

x = exponent = 2.0;

Couard [3] and Gant [4] developed this model in Europe during the thirties. More than fifty thousand field measurements were taken to calibrate the model.

2.2 Research Efforts by the Chessie System

During the late sixties and early seventies, research personnel on the Chessie System (C&0-B&O Railroads) undertook a project to review the Couard-Gant model and determine its suitability for North American applications. An extensive rail measurement program was initiated in main line territories that met the following criteria:

(a) Continuously welded rail, first position, weighing at least 115 lbs./yd.;

(b) Operating speeds between 40 and 60 MPH in tangents; and

(c) No more than two inches of unbalance in curve superelevation.

The measuring program comprised 1500 samples, usually taken at ten measurements per rail. The cumulative tonnage between successive measurements ranged between 50 and 300 MGT, depending on traffic density and curvature. After processing, the model was recalibrated and compared with the original. The comparison revealed that, in general, Chessie's wear rates were lower than those in Europe, and the effect of curvature was also less severe than the Europeans had experienced. Chessie research engineers identified three important reasons for differences between the two continents:

(1) The carbon content and resulting hardness were lower for European rails. Typical rail steel Brinell hardness was approximately 230, as compared with 260 for North American rail.

(2) There were significant numbers of two axle cars in Europe at the time when the model was first calibrated. As a percentage of static load, the dynamic loads were higher with two

wheeled equipment. On curves, the angle of attack (wheel flange relative to gage face of rail) is higher for two axle equipment, resulting in higher wear.

(3) In Europe, the unbalanced superelevation increased with the degree of curvature, whereas there was a constant unbalance in Chessie tracks. Wheel flange forces on the gage face of the high rail increase with the amount of unbalance.

The constants developed by the Chessie System are compared with the European experience in Table 1.

2.3 Wheel Loads as an Additional Variable

Improved productivity, throughout railroad history, has been associated with higher speeds and heavier unit loads. Traditionally, the load constraints have been reduced through use of larger components and/or improved quality in both cars and track. Maintenance of way officers, faced with accelerated degradation, countered with heavier rails, improved cross-sectional designs to lower the stresses at fillets, and eventually used continuous welded rail to reduce joint problems. While these developments were being made, a few far-sighted individuals began to document concerns about stresses in the wheel-to-rail contact areas. The accelerated deterioration of rail, predicted by these individuals, is now receiving extensive interest and documentation.

The first signs of these problems were observed in the early sixties when an ore carrying railroad, operating four axle cars of 90 tons capacity, found that the service life of their rail was drastically reduced [5]. Rail that had been in track for only six years (125 million gross tons accumulated traffic) exhibited accelerated wear, shelling and corrugations on curves. Of the greatest significance, however, was the deterioration of the running surface in tangent track. Many locations had head checks extending halfway across the rail head. At other locations, metal had spalled out across the entire contact surface. This rail was eventually replaced after carrying nearly 300 million gross tons, about one-half of its historical life expectancy.

An AAR laboratory test was conducted in 1965 to further quantify the effects of high contact pressures between rail and wheels, and the results are shown in Table 2 [6].

TABLE 1 — Comparison of Couard-Gant Model Equation Parameters for European and North American (Chessie System) Experience.

Constant	Represents	Calibration Values European	Calibration Values Chessie
a	wear rate of tangent rail (in.^{2}/MGT)	0.0007	0.00056
b	incremental wear of high rail in a curve (in.^{2}/MGT/(degree)^x	0.000173	0.000715
x	exponent	2.0	1.2

A quantitative evaluation of the effects of wheel load on rail life can be based on Barwell's statement [7]:

> "Experiment shows great fidelity to the expression that the number of load cycles to cause shelling failure is inversely proportional to the cube of the load which is, in turn, proportional to the ninth power of the Hertzian stress. . . . ."

Mathematically, the Barwell Equation can be expressed as:

$$N = A / W^3 \tag{2}$$

where:

N = number of load cycles to cause surface fatigue;

W = wheel load;

A = constant which varies with both operating characteristics and the rail steel properties.

The rail life, in terms of accumulated load, is then:

$$T = NW = A / W^2 \tag{3}$$

where:

T = total accumulated loads.

The authors believe that a relationship between wear rates and wheel loads can also be described using the Barwell equation, because an important component of wear is plastic deformation resulting from the high contact stresses which induce shelly failures. Comparisons of empirical wear estimates and actual results, discussed in Section 3.1, have confirmed this relationship.

TABLE 2 — Laboratory Test Results Relating Wheel Load and Diameter to Service Life of Rail.

	Millions of Cycles to Failure	
Wheel Load (Kips)	**33-Inch Wheel Diameter**	**38-Inch Wheel Diameter**
25.0	3.74	5.27
27.5	3.57	3.40
30.0	2.01	2.93
32.0	1.51	1.77
35.0	1.21	1.29

Using both the Barwell and Chessie Equations, the authors have been able to develop the following rail wear model:

$$L_c = \frac{TW_c}{K_c[Ta(P/23)^2 + TbD^x(P/23)^y](1 + dG^2) + c_c} \tag{4}$$

For tangent track, the equation simplifies to:

$$L_t = \frac{TW_t}{K_t\, Ta(P/23)^2(1 + dG^2) + c_t} \tag{5}$$

where: K_t, K_c = constants (1.00 for standard carbon rail and no lubrication);

L_t = life of tangent rail (MGT);

L_c = life of curved rail (MGT);

W_t = allowable cross-sectional area loss of rail in tangent track (in.^{2});

W_c = allowable cross-sectional area loss of rail in curves (in.^{2});

T = annual traffic density (MGT/yr.);

D = curvature (degrees);

G = grade (percent);

P = static wheel load (kips);

a = tangent track wear rate (in.^{2}/MGT) = 0.00056;

b = incremental wear rate for the high rail in curves (in.^{2}/MGT/(degree)^x) = 0.000715;

c_t = corrosion constant for tangent track (in.^{2}/yr.) = 0.0045;

c_c = corrosion constant for curved track (in.^{2}/yr.) = 0.0052;

y = modified Barwell exponent for curvature = (3.70 - 0.40D) when D≥2°, and (2.40 - 0.14D) when D>5°;

x = exponent for curvature = 1.30;

d = track gradient constant (1/percent^2) = 0.023.

It can be seen that the modified equation predicts the following:

1. Exponential increases in rail wear for wheel loads over 23 kips;
2. Modestly increased influence of the degree of curvature as compared to the Chessie modified version of the Couard-Gant expression (x = 1.3 versus x = 1.2 for Chessie);
3. Reduced influence of vertical wheel loads on curve wear, as the curvature increases.

3.0 RESULTS

3.1 Comparison with Actual Wear Data

Figure 3 shows rail wear life of a 132RE section as a function of curvature, using a 25% head loss as a condemning limit. Four load spectra have been plotted and denoted by the average wheel load, in thousands of pounds (kips). The plot of 17, 20 and 23 kip loads represents a band in which one might reasonably expect wear from mixed traffic to fall. Also plotted are results from the first Heavy Hauls Railway Conference [8] and Hay's work for the Burlington Northern [9]. Of particular note is the agreement of the mineral carrying roads, involving 100-ton cars, and the model results in the range from two to six degrees. Professor Hay's results from mixed traffic appear to correlate below seven degrees. Hay predicts a greater influence due to curvature at higher degrees. Little is known about the wheel load spectra, or the operating policies from which these data were derived.

The wear rate predicted by the model for five degrees of track curvature is approximately 0.012 inches ^{2}/MGT, compared to 0.013 for FAST results [10], a discrepancy of approximately 8%. Considering that FAST operates at three inches of unbalanced superelevation, wear rates should be greater.

Rail wear lives for specific wheel loads have been calculated and are shown as information in Figure 4. The reduction in rail life with increasing wheel load is clearly shown. As curvature increases, the influence of vertical wheel load diminishes.

Figure 5 shows the scatter of actual data about the predicted wear lives. Very little data exists for curves over 7 degrees, because few railroads have a significant proportion of their trackage in this range of curvature.

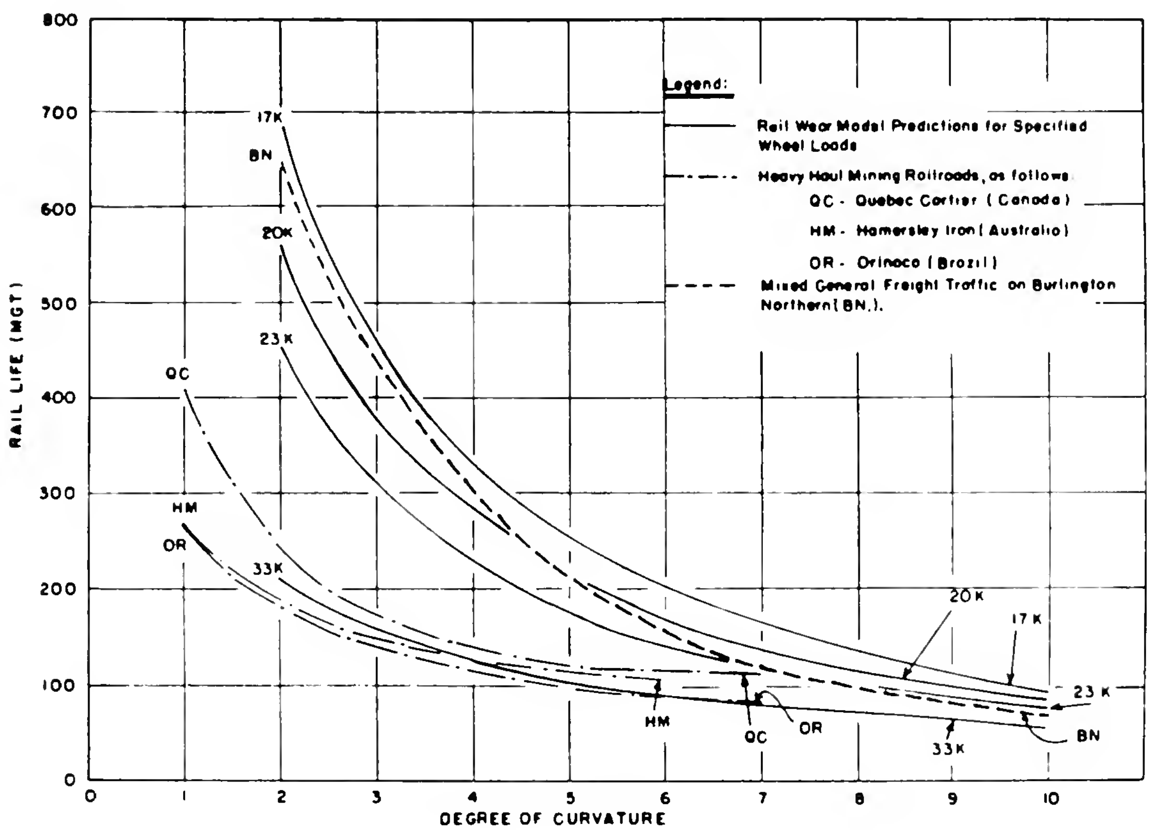

FIG. 3. Actual and Theoretical 132RE Standard Carbon Rail Life as Functions of Curvature and Wheel Load. A Rail Head Area Loss of 25% Represents the Condemning Wear Limit.

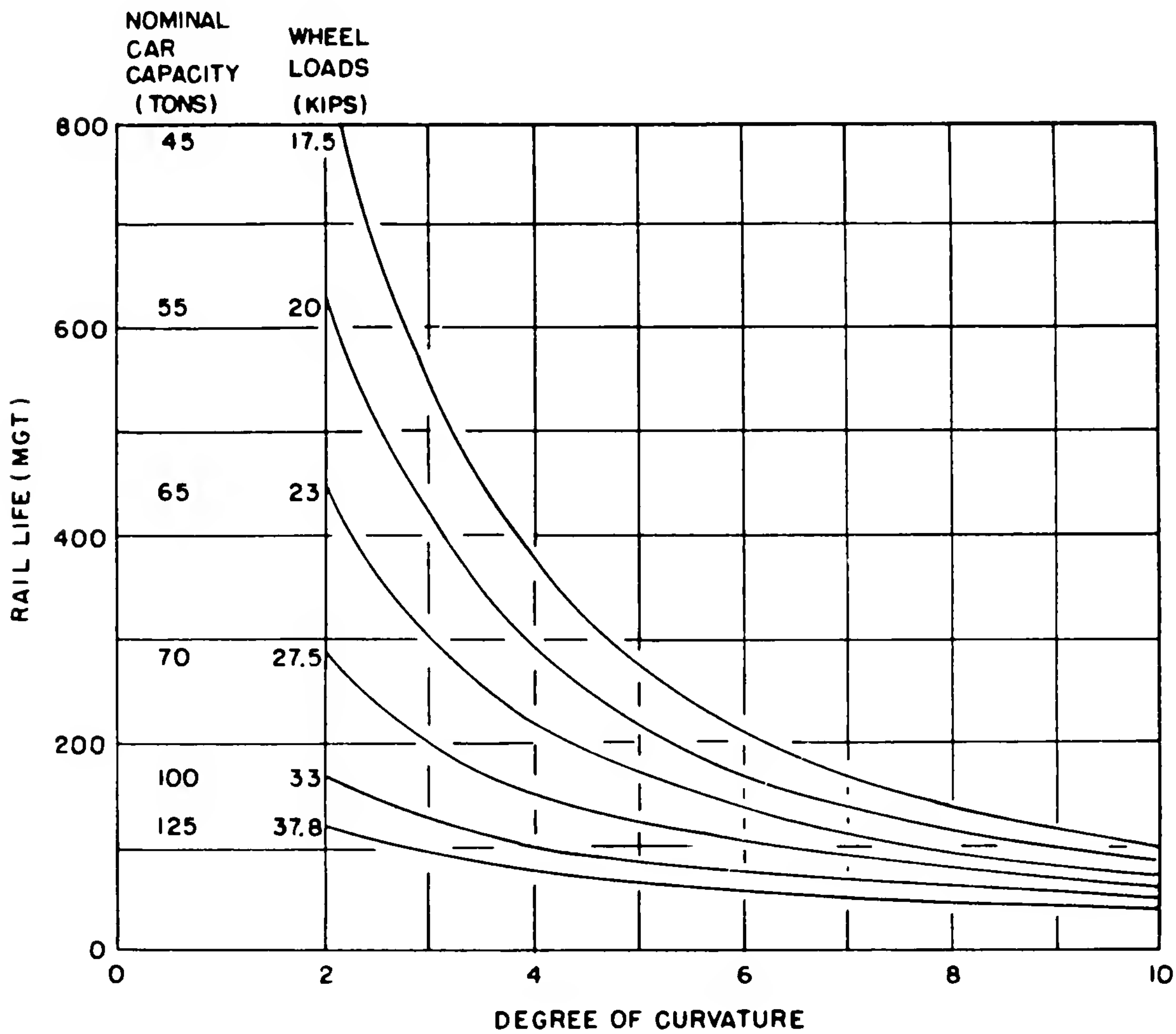

FIG. 4. Theoretical 132RE Standard Carbon Rail Life as Functions of Track Curvature and Wheel Load. A Rail Head Area Loss of 25% Represents the Condemning Wear Limit.

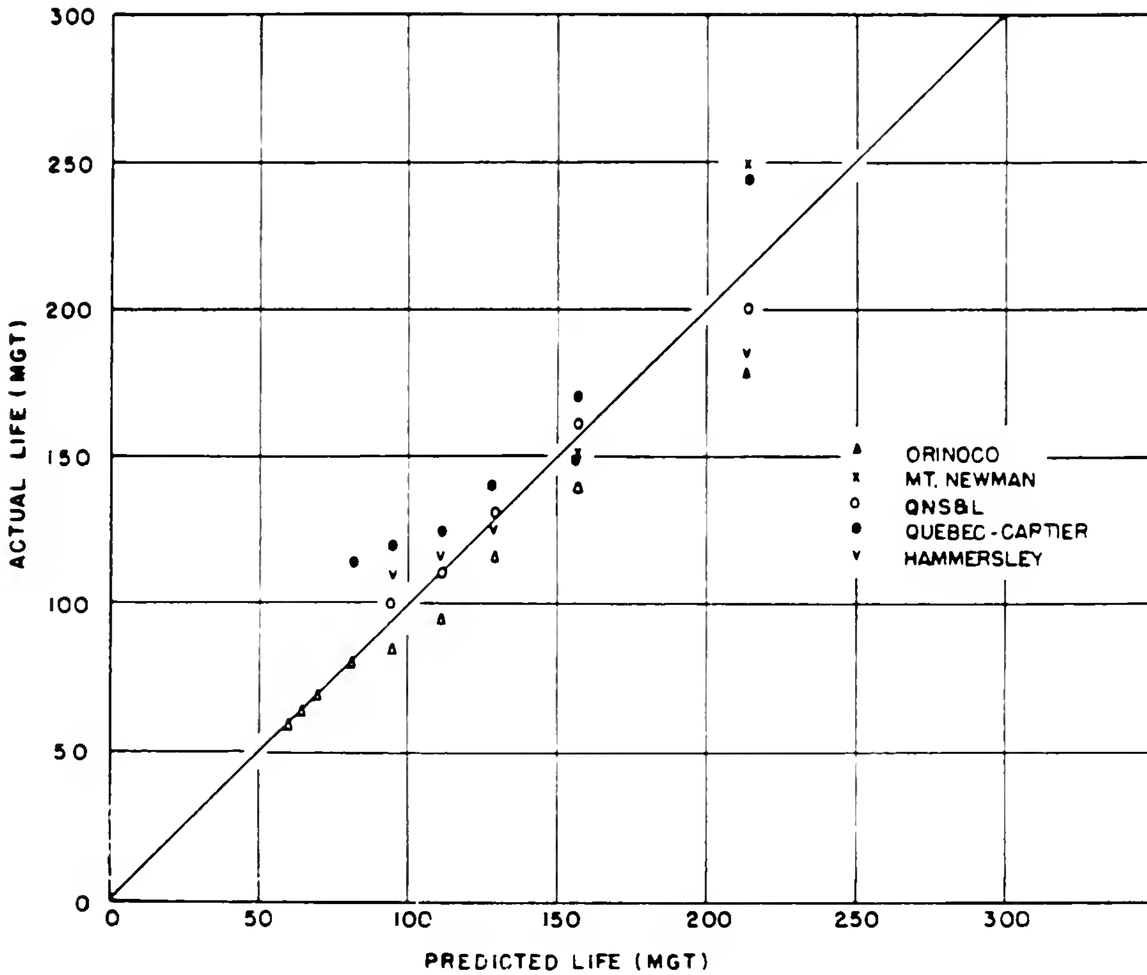

FIG. 5. Scatter Diagram Showing Actual Versus Predicted Rail Wear Lives.

The calibration of the Chessie model from mixed traffic wear data, and the close agreement of the later version with the closely controlled FAST environment and the Heavy Haul Roads, shows that the model gives results within 15 percent of actual experience.

3.2 Model Calculations for Known Wheel Loads

Increased computer usage in the rail industry is making improved statistical data available. The reporting systems on several railroads now give either specific car weights by route, or for the entire system. From this type of information, wheel load adjustments can be calculated for specific routes, or for an overall system average. The computerization of such calculations is not absolutely necessary, but would reduce the time required to make them.

The best approach to handle assorted wheel loads is to group them into categories, as shown in Table 3.

The Rail Wear Equation (5) for tangent track can then be rewritten as follows:

$$L_t = \frac{(T_1 + T_2 + \cdots T_8)W_t}{K_t a[T_1(P_1/23)^3 + T_2(P_2/23)^2 + \cdots + T_8(P_8/23)^2](1 + dG^2) + c_t} \tag{6}$$

where:

T_i = annual traffic density in category i (MGT/yr.);

P_i = static wheel load in category i (kips);

A similar modified Equation (4) would be used for curves.

TABLE 3 — Sample Categorization of Wheel Loads.

Category Number	Wheel Load Range (Kips)	Average Wheel Load	Tonnage Carried in Category
1	0 - 10	7.5 (P_1)	T_1
2	10 - 15	12.5 (P_2)	T_2
3	15 - 20	17.5 (P_3)	T_3
4	20 - 25	22.5 (P_4)	T_4
5	25 - 30	27.5 (P_5)	T_5
6	30 - 35	32.5 (P_6)	T_6
7	35 - 40	37.5 (P_7)	T_7
8	40+	42.5 (P_8)	T_8

Σ T = Total Tonnage

3.3 Sample Calculations for System Loads and Fractional Tonnages

If it is desired to calculate a more generalized wear rate, e.g., system rail life, and the actual distribution of loads is not known, other information may be substituted. Most roads, for example, know their system-wide average car loadings, percentages of loaded and empty car miles, and locomotive miles. This information can be derived from the R-1 Report, ** and typical values are shown in Table 4.

The "average" load may then be calculated as follows:

For Loaded cars:	
(0.58)(72+30) = (102)(0.58)	= 59.2 tons
For Empty cars:	
(0.42)(30)	= 12.6 tons
For Locomotives:	
(1.00)(0.053)(150)	= 8.0 tons
Total	79.8 tons

After the "average" load has been calculated, Table 5 can then be constructed for calculating the P_i's and T_i's in Equation (6).

The average static wheel loads and fractions of total tonnage can then be substituted into Equation (6), along with the number of MGT's for the line segment in question. This procedure would be expected to calculate rail head wear rates that average to within 15 percent of actual wear rates, unless line-specific traffic values differ significantly from the system average. This approach has the advantage of calculating values for a wide variety of line segments, whereas the approach using known wheel loads would calculate more accurate values on a line-specific basis.

**The R-1 Report is an annual report filed by Class 1 common carrier railroads with the Interstate Commerce Commission.

TABLE 4 — Typical Railroad Traffic Data.

Fraction of loaded car-miles:	58%
Fraction of empty car-miles:	42%
Average car lading weight:	72 tons
Average car light weight:	30 tons
Locomotive miles per total car miles	0.053
Average weight of locomotive (mix of 4 and 6 axle units)	150 tons

TABLE 5 — Fraction of Total Tonnage for Loaded Cars, Empty Cars and Locomotives.

Average Load Type	Weight (Tons)	Average Static Wheel Load (Kips)	Fraction of Total Tonnage
Loaded Car	102	25.5	59.2 / 79.8 = 0.742
Empty Car	30	7.5	12.6 / 79.8 = 0.158
Locomotive	150	32.0	8.0 / 79.8 = 0.100

3.4 Discussion of Wear Sensitivity

Much work remains to be done in adequately documenting all of the factors that are associated with wear at the wheel-rail interface. This Rail Wear Model will give satisfactory results when used under the following conditions:

1. Continuous welded rail, in first position, weighing at least 115 lbs/yard;
2. Operating speeds between 40 and 60 MPH in tangent track;
3. No more than two inches of unbalanced superelevation in curves.

Estimating the wear on lighter weights of rail would probably be difficult without additional calibrations. Light rail typically experiences low traffic densities, and the corrosion loss can be a significant portion of the total wear. The corrosion constant which is based on Chessie experience might be expected to be different on other roads. For higher annual tonnages this factor is not significant, because traffic-induced wear predominates. Errors introduced by using the model at speeds below 40 MPH and above 60 MPH will probably not be significant. High unbalanced superelevations would lead to significant model errors. Care should also be taken wherever actual train speeds deviate from design speeds. If a 3° curve has 3.5 inches of superelevation, it is permissible to operate at 50 MPH with 1.5 inches of unbalance. If many trains have to stop on or near the curve, the effective unbalance will be significantly changed. High rail wear would be reduced and the low rail wear correspondingly increased.

The wear model was calibrated for freight traffic, and would require recalibration for use with transit-type passenger loads, which are typically smaller in magnitude but much higher in frequency. Other factors, such as extreme curvature and frequency of powered axles, would also affect results.

Certain other assumptions in the model deserve discussion. No heat-treated or special alloy rail was involved in the calibration of this model. The predicted wear rates, therefore, apply only to standard carbon rail and would have to be adjusted for cases involving premium rail. Modification could be made to the "K" factors in the wear equation. As an example, the FAST results have shown a wear reduction of approximately 50 percent for heat-treated rail ($K_c = 0.50$ in Equation (4) on 5^o curves [10]).

The sampling techniques used to calibrate the Chessie Model also imply that "average" standard carbon rail is represented in the equation. In reality, rail chemistries vary, and published FAST reports have already shown a 30-50 percent variance in gage face wear for standard carbon rail, due to variance in chemistry within approved AREA-specified limits. The literature shows a distinct relationship between hardness and the relative percentages of carbon, manganese, and silicon [10]. If wear rates for rail with a specific chemistry were to be projected, the "K" factors could be adjusted using the relationships developed at FAST.

Wear information used in the Chessie study contained little or no data on the effects of rail lubrication. At a RSMA-AAR-AREA symposium on rail wear and lubrication in Memphis [11], there seemed to be a consensus that generous rail lubrication reduced wear rates from 7 to 10 times. The effects on premium rails are less spectacular, but still significant. Lubrication, as practiced in the field, is far from an exact science and it is doubtful that maximum results could be achieved in practice. The effects of lubrication will also vary, depending on the track curvature and rail metallurgy.

Poor cross level and track surface will cause increased flange contact with rails and, therefore, higher dynamic forces. Higher, non-uniform wear rates could be expected in these situations.

Preliminary results indicate that the wear rates on steep, i.e., over one percent, gradients may exceed those predicted by the model, especially where heavy sanding occurs. Factors influencing the rate of wear include increased tractive effort, higher coefficients of friction, and possibly wheel/rail creep.

In summary, care should be exercised in applying the model where "abnormal" conditions exist. With the use of FAST and other results, model modifications are possible for special cases by means of adjustments to constants K_t and K_c. The overall simplicity of the wear equation also permits wear sampling by individual users in order to validate factors on a site-specific basis.

4.0 RECOMMENDATIONS FOR FUTURE WORK

With the numerous variables affecting rail wear, the collection of more data under controlled conditions would be very helpful. Much more data will be required if factors, such as gradient, lubrication, rail chemistry and superelevation unbalance, are to be included in future calibrations. The inclusion of such factors would broaden the applicability of the model to specific cases. Perhaps just as important as improving the empirical wear model, additional data will also be required for the future development of more comprehensive theoretical wear models. Since these data take time to collect and analyze, there would be no better time to start than right now!

Additional work is also required to relate the changes in horizontal and vertical wear dimensions to the head area loss caused by wear.

The relationships between static and dynamic loadings, as functions of speed, track irregularities, and equipment defects, e.g., wheel flats, need to be more clearly defined.

Additional wear data from lower tonnage lines and light rail sections would increase user confidence in the wear model for estimating second and third position rail life.

5.0 REFERENCES

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6.0 APPENDIX

This section contains a sample computer program to make the rail wear calculations that have been discussed in this paper. The basic program involves a main program that calls five subroutines, as shown in Figure 6. Much of the sample program listing is concerned with input and output formats. This particular program is designed to read a file that lists curves, and then calculates the rail life due to wear for each curve. Since wheel loads have a major impact on wear, data on the static wheel load distributions can be inputted, and the program can convert that information into fractions of tonnages, as shown in Section 3.3. Subroutine CWWEAR contains the necessary steps to calculate the rail wear for a series of curves that have been previously inputted.

Included after the listing of the program are sample inputs, showing the correct order of the input data. A sample file containing a typical list of curves is also provided. The last page shows the expected output, which includes an echo of the input data, along with the list of curves and predicted rail lives for each curve. This program will make calculations for curvatures in the range of 2 to 10 degrees, ignoring anything less than 2 degrees and above 10 degrees.

AAR COMPUTER MODEL FOR REINER AND STAPLIN'S EMPIRICAL RAIL WEAR MODEL

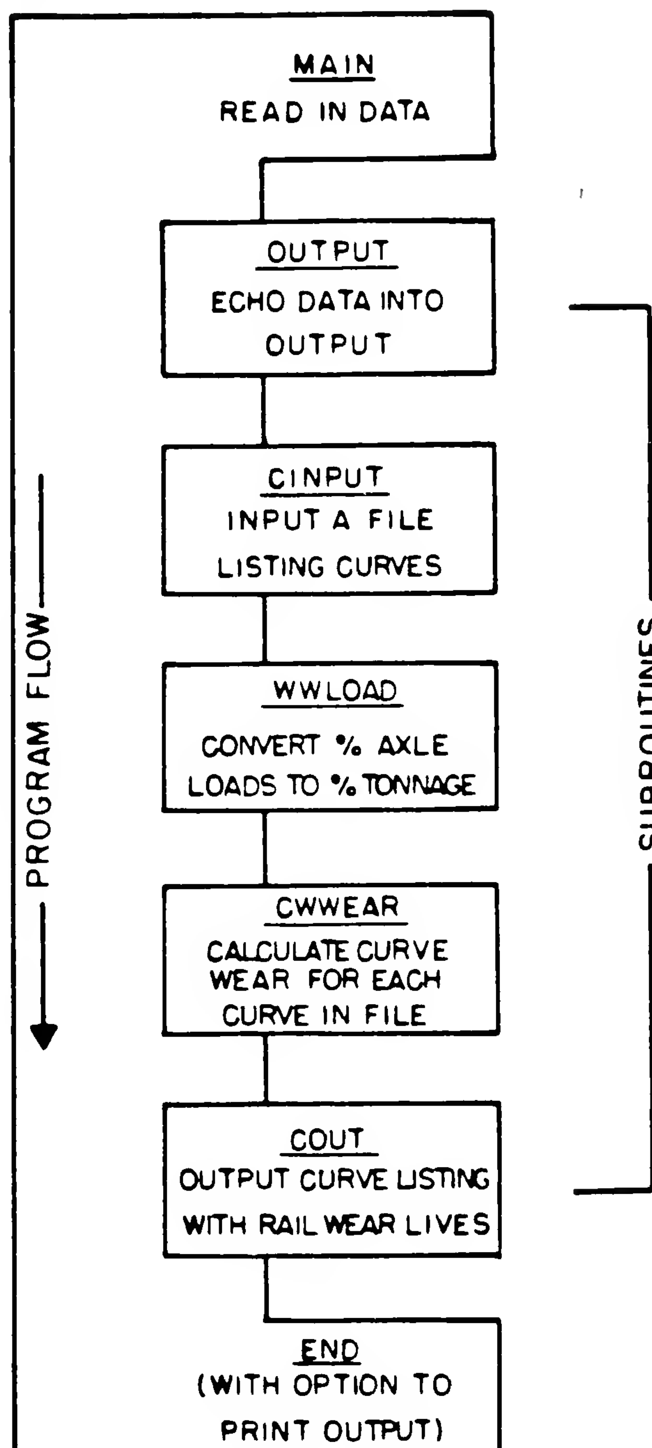
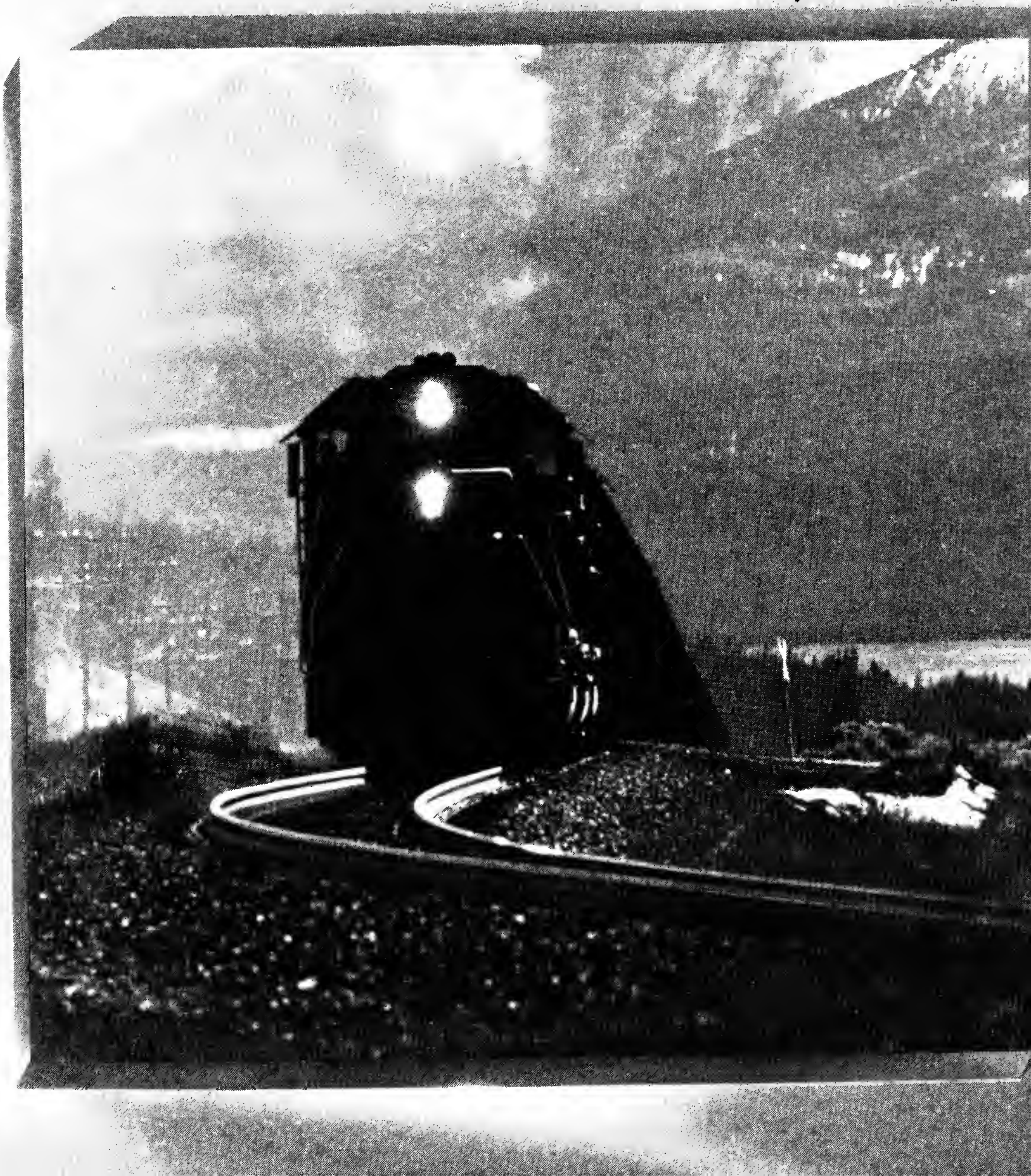


FIG. 6. Flow Diagram of the Computer Program for Evaluating the Empirical Rail Wear Formula.

6.A Program Listing

6.A.1 MAIN Program

```
C * * * * * * * * * * * MAIN PROGRAM * * * * * * * * * * * * * * * * * *
C
C   PURPOSE: TO COMPUTE CURVE RAIL LIFE FOR A LISTING OF CURVES, USING
C            REINER AND STAPLIN'S EMPIRICAL RAIL WEAR MODEL.
C
C   CALLS   : OUTPUT - ECHO OF INPUTS
C            CINPUT - READS FILE CONTAINING A CURVE LISTING
C                     (UP TO 70 CURVES)
C            WWLOAD - CONVERTS % AXLE LOADS TO % TONNAGE
C            CWWEAR - CALCULATES CURVE WEAR USING THE REINER AND
C                     STAPLIN FORMULA
C            COUT   - OUTPUTS CURVE LISTING AND CURVE WEAR LIVES
C
C   AUTHORS: L. M. LEE AND T. R. WELLS
C            ASSOCIATION OF AMERICAN RAILROADS, 1982.
C
C   NOTES   : THIS PROGRAM IS TAKEN FROM THE RAIL PERFORMANCE MODEL,
C            WHERE YEARLY WEAR RATES FOR INDIVIDUAL CURVES ARE NEEDED.
C            SEE SUBROUTINES FOR VARIABLE DEFINITIONS.  THIS FORTRAN
C            PROGRAM WORKS INTERACTIVELY ON THE AAR'S DEC 2050 SYSTEM.
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
      DIMENSION TCWEAR(70),CWEAR(70),PTS(70),PSTT(70),
     .          DEGREE(70),XXLOAD(11),DIST(11)
      DIMENSION TLOAD(11),TITLE(12)
      REAL*8 INFIL
50    OPEN(UNIT=27,ACCESS='SEQOUT',FILE='WEAR.LST',DEVICE='DSK')
20    FORMAT(F,I,6F)
21    FORMAT(A10,12A5)
22    FORMAT(I)
31    FORMAT(2F)
100   FORMAT(A1)
      READ(5,20)HEAD,IWT,YRTON,CNDEM,GRADE,WRCON2,A,B
      READ(5,21)INFIL,(TITLE(K1),K1=1,12)
      READ(5,22)II
      DO 7 K=1,II
      READ(5,31)XXLOAD(K),DIST(K)
7     CONTINUE
      CALL OUTPUT(IWT,YRTON,CNDEM,GRADE,WRCON2,A,B,INFIL,
     .DIST,XXLOAD,II,TITLE)
      CALL CINPUT(DEGREE,NOC,PTS,PSTT,INFIL)
      CALL WWLOAD(XXLOAD,DIST,II,TLOAD)
      CALL CWWEAR(YRTON,GRADE,XXLOAD,TLOAD,II,NOC,DEGREE,CWEAR,
     .             WRCON2,A,B)
      CALL COUT(HEAD,CWEAR,PTS,PSTT,DEGREE,NOC,CNDEM,IWT,YRTON)
      WRITE(5,30)
30    FORMAT(//' PRINTOUT REQUIRED FOR CURVE SECTION FILE?--->',$)
      READ(5,100)PP
      IF(PP.EQ.'Y')CLOSE(UNIT=27,DISPOSE='LIST')
      IF(PP.EQ.' '.OR.PP.EQ.'N')CLOSE(UNIT=27)
      STOP
      END
```



6.A.2 Subroutine OUTPUT

```
C * * * * * * * * * * * SUBROUTINE OUTPUT * * * * * * * * * * * * * *
C
C   PURPOSE:  ECHO INPUT DATA ONTO OUTPUT FILE
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
      SUBROUTINE OUTPUT(IWT,YRTON,CNDEM,GRADE,WRCON2,A,B,INFIL,DIST,
     . XXLOAD,II,TITLE)
      REAL DAY(2),IHOUR
      DIMENSION TITLE(1),XXLOAD(1),DIST(1)
      REAL*8 INFIL
      CALL DATE(DAY)
      CALL TIME(IHOUR)
      WRITE(27,5)(DAY(J),J=1,2),IHOUR
5     FORMAT(50X,11HDATE OF RUN,3X,2A5/
     . 50X,11HTIME OF RUN,3X,A5,3H:00///)
C * * VARIABLE 'TITLE' IS FOR DESCRIPTION/IDENTIFYING OUTPUT
      WRITE(27,10)(TITLE(J),J=1,12)
10    FORMAT(
     . 6X,12A5//
     . 6X,12(2H* ),20H ECHO OF INPUT DATA ,11(2H *)//)
11    FORMAT(
     . 6X,20HRAIL WEIGHT (LBS/YD),42X,I4/
     . 6X,28HANNUAL TRAFFIC DENSITY (MGT),26X,F12.4/
     . 6X,53HCONDEMNING LIMIT OF RAIL HEAD (FRACTION OF RAIL HEAD),X,
     .F12.4/
     . 6X,30HGRADIENT OF TRACK (PERCENTAGE),24X,F12.4/
     . 6X,49HCORROSION CONSTANT FOR CURVE(INCHES**2/YEAR),5X,F12.6/
     . 6X,38HTANGENT WEAR RATE (INCHES SQUARED/MGT),16X,F12.6/
     . 6X,57HINCREMENTAL CURVE HIGH RAIL WEAR RATE(INCHES**2/MGT),
     .F9.6/
     . 6X,18HCURVE SEGMENT FILE,36X,A10//)
40    FORMAT(6X,'WHEEL LOADS (KIPS)',3X,'PERCENTAGE')
51    FORMAT(8X,F10.4,6X,F10.3)
      WRITE(27,11)IWT,YRTON,CNDEM,GRADE,WRCON2,A,B,INFIL
      WRITE(27,40)
      DO 50 I=1,II
      WRITE(27,51)XXLOAD(I),DIST(I)
50    CONTINUE
      WRITE(27,70)
70    FORMAT(6X,66('*')//)
18    RETURN
      END
```

6.A.3 Subroutine CINPUT

```
C * * * * * * * * * * * SUBROUTINE CINPUT * * * * * * * * * * * * * *
C
C  PURPOSE:    TO OPEN AND READ AN OUTSIDE FILE LISTING CURVATURE AND
C              POINTS OF TANGENT TO SPIRAL, SPIRAL TO TANGENT, ETC.
C
C              VARIABLE     DESCRIPTION
C  INPUTS :    INFIL        NAME OF FILE CONTAINING CURVE DATA
C
C  OUTPUTS:    DEGREE       DEGREE OF EACH CURVE
C              NOC          LENGTH OF CURVE FILE (NUMBER OF CURVES IN
C                           SEGMENT)
C              PTS          POINT OF TANGENT TO SPIRAL (MILE POST)
C              PSTT         POINT OF SPIRAL TO TANGENT (MILE POST)
```

```
C
C  OTHER
C  PARAMETERS:  CMINT    MINUTES (SIXTIETHS OF DEGREE)
C               SUPEL    SUPELEVATION OF THE CURVE
C               PTS      POINT OF TANGENT TO SPIRAL (MILE POST)
C               PSC      POINT OF SPIRAL TO CURVE (MILE POST)
C               PCSS     POINT OF CURVE TO SPIRAL (MILE POST)
C               PSTT     POINT OF SPIRAL TO TANGENT (MILE POST)
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
      SUBROUTINE CINPUT (DEGREE,NOC,PTS,PSTT,INFIL)
      DIMENSION DEGREE(70),PTS(70),PSTT(70)
      REAL*8 INFIL
      OPEN(UNIT=22,ACCESS='SEQIN',FILE=INFIL)
2     FORMAT(2F5.2,5F10.6)
      I=0
18    I=I+1
      READ(22,2,ERR=26)XDEG,CMINT,SUPEL,PTS(I),PSC,PCSS,PSTT(I)
      XDEG=ABS(XDEG)
C * * VARIABLE SUPEL IS NOT USED IN THIS ALGORITHM
C * * LENGTH OF CURVE IS CALCULATED FROM POINT OF TANGENT TO SPIRAL
C * * TO SPIRAL TO TANGENT WHEN SPIRALS ARE PRESENT
      IF(PTS(I).EQ.0.0)PTS(I)=PSC
      IF(PSTT(I).EQ.0.)PSTT(I)=PCSS
C * * CHECK TO SEE IF CURVATURE INCLUDES FRACTIONAL DEGREES
      TEST=XDEG-IFIX(XDEG)
      IF(TEST.GT.0.0)GO TO 24
C * * CONVERT MINUTES INTO FRACTIONAL DEGREES AND ADD TO DEGREES
      XDEG=XDEG+(CMINT/60.)
24    DEGREE(I)=XDEG
      IF(DEGREE(I).LT.2.)GO TO 18
      GO TO 18
26    NOC=I-1
      CLOSE(UNIT=22)
      END
```

6.A.4 Subroutine WWLOAD

```
C * * * * * * * * SUBROUTINE WWLOAD * * * * * * * * * * * * * * *
C
C   PURPOSE:   TO CONVERT PERCENTAGE OF WHEEL LOADS INTO FRACTIONAL
C              TRAFFIC TONNAGE.
C
C              VARIABLE    DESCRIPTION
C   INPUTS :   XXLOAD     WHEEL LOADS (KIPS)
C              PERC       PERCENT OF TOTAL WHEEL LOADS
C              II         NUMBER OF DATA LINES OF LOAD SPECTRA ARRAY
C
C   OUTPUTS:   TTLOAD    ARRAY OF TONNAGE FRACTION FOR EACH WHEEL LOAD
C
C
C   NOTE   :   LENGTHS OF ARRAYS ARE ASSUMED TO HAVE A MAXIMUM OF 11
C              SHOWN IN XXLOAD(11),PERC(11),TLOAD(11),WPL(11)
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
      SUBROUTINE WWLOAD(XXLOAD,PERC,II,TLOAD)
      DIMENSION WPL(11),TLOAD(11),XXLOAD(1),PERC(1)
     TOT3=0.0
     DO 29 J=1,II
```

```
           WL=XXLOAD(J)
           PL=PERC(J)/100.
           WPL(J)=WL*PL
           TOT3=WPL(J)+TOT3
   29      CONTINUE
           DO 18 J=1,II
           TLOAD(J)=WPL(J)/TOT3
  18       CONTINUE
            RETURN
            END
```

6.A.5 Subroutine CWWEAR

```
C * * * * * * * * * * * SUBROUTINE CWWEAR * * * * * * * * * * * * * *
C
C   PURPOSE: TO CALCULATE YEARLY RAIL WEAR FOR INDIVIDUAL CURVES
C
C            VARIABLE       DESCRIPTION
C   INPUTS : YERTON       ANNUAL TRAFFIC TONNAGE
C            GRADE        GRADIENT OF TRACK (PERCENT)
C            XXLOAD       WHEEL LOADS COMBINATION
C            TLOAD        FRACTIONAL TRAFFIC TONNAGE PER WHEEL LOAD
C            II           NUMBER OF DATA LINES IN XLOAD, TLOAD
C            NOC          LENGTH OF CURVE FILE (NUMBER OF CURVES IN
C                         SEGMENT)
C            DEGREE       DEGREE OF CURVE
C            WRCON2       CURVE WEAR CONSTANT
C            K2           CONSTANT
C            A            TANGENT WEAR RATE (INCHES SQUARE PER MILE)
C            B            INCREMENTAL CURVE HIGH RAIL WEAR RATE
C
C   OTHER
C   PARAMETER: GRAC       GRADIENT OF TRACK CONSTANT
C                EXPON    WHEEL LOAD EXPONENT
C
C   OUTPUT   : CWEAR      YEARLY CURVE WEAR LOSS (INCHES SQUARE)
C
C * * * * * * * * * ** * * * * * * * * * * * * * * * * * * * * * * *
C
            SUBROUTINE CWWEAR(YERTON,GRADE,XXLOAD,TLOAD,II,NOC,
         .                   DEGREE,CWEAR,WRCON2,A,B)
            DIMENSION DEGREE(70),CWEAR(70),XXLOAD(1),TLOAD(1),
         .            GRAENT(70)
            DATA K2,GRAC,EXPON/1,0.023,2./
            DO 20 K=1,NOC
            WLFAC1=0.0
            WLFAC2=0.0
C * * TEST FOR CURVE DEGREE BEYOND 2 - 10 DEGREES LIMITS  MODEL
            IF(DEGREE(K).GE.2.0.AND.DEGREE(K).LE.10.)GO TO 5
            CWEAR(K)=0.0
            GO TO 20
C * * SET PARAMETERS
5           GRAENT(K)=GRADE
            YM=3.7-(0.4*DEGREE(K))
            XM=1.3
            IF(DEGREE(K).GT.5)YM=2.4-(0.14*DEGREE(K))

C * * FIRST STEP IS TO SUM UP WHEEL LOAD EFFECTS
            DO 10 J=1,II
            P1LOAD=(XXLOAD(J)/23.)**EXPON
```

```
          P2LOAD=(XXLOAD(J)/23.)**YM
          WLFAC1=WLFAC1+P1LOAD*TLOAD(J)
          WLFAC2=WLFAC2+P2LOAD*TLOAD(J)
   10     CONTINUE
C * * GRADIENT FACTOR CALCULATION
          T=1.0+(GRAC*(GRAENT(K)**2))
C * * ANNUAL TANGENT RAIL WEAR CALCULATION(INCHES SQUARE PER YEAR)
          WLFAC3=WLFAC1*A*YERTON
          WLFAC4=WLFAC2*(DEGREE(K)**XM)*B*YERTON
          CWEAR(K)=K2*(WLFAC3+WLFAC4)*T+WRCON2
   20     CONTINUE
          RETURN
          END
```

6.A.6 Subroutine COUT

```
C * * * * * * * * * * * * * * SUBROUTINE COUT * * * * * * * * * * * * * *
C
C  PURPOSE: TO OPEN AN OUTPUT FILE FOR CURVE DATA
C
C                  VARIABLES      DESCRIPTION
C  INPUTS    :      PTS           POINT OF TANGENT TO SPIRAL (MILE POST)
C                   PSTT          POINT OF SPIRAL TO TANGENT (MILE POST)
C                   DEGREE        DEGREE OF CURVE
C                   NOC           LENGTH OF CURVE FILE
C                   IWT           RAIL WEIGHT (LBS/YD.)
C                   CNDEM         CONDEMNING LIMIT OF RAIL HEAD (%)
C                   HEAD          RAIL HEAD AREA (SQUARE INCHES)
C
C  OUTPUTS   :      FLIFE         CURVE LIFE IN TERMS OF YEARS
C                   TONN          CURVE LIFE IN TERMS OF TONNAGE
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
          SUBROUTINE COUT(HEAD,CWEAR,PTS,PSTT,DEGREE,NOC,CNDEM,
     . IWT,YRTON)
          DIMENSION PTS(70),PSTT(70),DEGREE(70),CWEAR(70)
   13     FORMAT(5X,'MP (FROM)',3X,'MP (TO)',3X,'CURVATURE',5X,
     .'HEAD',6X,' RAIL LIFE ',
     ./,28X,'DEGREE',5X,'WEAR RATE',4X ,I3,X,'LBS/YD'/,
     .39X,'(IN**2/YR.)',5X,'YEARS',6X,'(MGT)'/5X,68('-'))
   11     FORMAT(5X,2F9.3,2X,F8.4,7X,F8.4,6X,F5.2,3X,F10.2)
   19     FORMAT(5X,2F9.3,2X,F8.4,12X,'*',11X,'*',11X,'*')
          WRITE(27,13)IWT
          DO 7 K=1,NOC
          IF(CWEAR(K).NE.0.)GO TO 9
          WRITE(27,19)PTS(K),PSTT(K),DEGREE(K)
          GO TO 7
    9     FLIFE=HEAD*CNDEM/CWEAR(K)
          TONN=FLIFE*YRTON
          WRITE(27,11)PTS(K),PSTT(K),DEGREE(K),CWEAR(K),FLIFE,TONN
    7     CONTINUE
          RETURN
          END
```

6.B Sample Program Input

```
                                                       Comments:
RUN PTWEAR                                             ¦Start program
4.32,119,30,0.25,0,0.0052,0.00056,0.000715             ¦See Line 3400 of MAIN
SCURVE.DAT EXAMPLE RUN AND OUTPUT FOR AAR REPORT ¦Curve data file and title
4                                                      ¦Number of wheel load/
10 25                                          \          percentage data pairs
20 35                                           ¦Static wheel load (kips)
30 35                                           ¦    and percent of total
40 5                                            ¦    wheels (total=100)
PRINTOUT REQUIRED FOR CURVE SECTION FILE?--->N

STOP

END OF EXECUTION
CPU TIME: 0.70  ELAPSED TIME: 1:49.36
EXIT
```

Sample Curve Data File (Named SCURVE.DAT)

```
0.00030.000.00000000000.000000001.000000002.000000000.000000
1.00000.000.00000000000.000000002.000000003.000000000.000000
1.00030.000.00000000000.000000003.000000004.000000000.000000
2.00000.000.00000000000.000000004.000000005.000000000.000000
2.00030.000.00000000000.000000005.000000006.000000000.000000
3.00000.000.00000000000.000000006.000000007.000000000.000000
3.00030.000.00000000000.000000007.000000008.000000000.000000
4.00000.000.00000000000.000000008.000000009.000000000.000000
4.00030.000.00000000000.000000009.000000010.000000000.000000
5.00000.000.00000000000.000000010.000000011.000000000.000000
5.00030.000.00000000000.000000011.000000012.000000000.000000
6.00000.000.00000000000.000000012.000000013.000000000.000000
6.00030.000.00000000000.000000013.000000014.000000000.000000
7.00000.000.00000000000.000000014.000000015.000000000.000000
7.00030.000.00000000000.000000015.000000016.000000000.000000
8.00000.000.00000000000.000000016.000000017.000000000.000000
8.00030.000.00000000000.000000017.000000018.000000000.000000
9.00000.000.00000000000.000000018.000000019.000000000.000000
9.00030.000.00000000000.000000019.000000020.000000000.000000
10.0000.000.00000000000.000000020.000000021.000000000.000000
10.0030.000.00000000000.000000021.000000022.000000000.000000

STRUCTURE OF CURVE FILE
ATTRIBUTE DEG REAL COL 1 5
ATTRIBUTE MNT REAL COL 6 10
ATTRIBUTE SUPER REAL COL 11 20
ATTRIBUTE PTS REAL KEY COL 21 30
ATTRIBUTE PSC REAL KEY COL 31 40
ATTRIBUTE PCS REAL KEY COL 41 50
ATTRIBUTE PST REAL KEY COL 51 60
ATT EQLAST TEXT COL 61 80
```

6.C Sample Program Output

```
                                              DATE OF RUN    2-Mar-83
                                              TIME OF RUN    13:09:00


 EXAMPLE RUN AND OUTPUT FOR AAR REPORT

* * * * * * * * * * * * *  ECHO OF INPUT DATA  * * * * * * * * * * * *


RAIL WEIGHT (LBS/YD)                                                119
ANNUAL TRAFFIC DENSITY (MGT)                                    30.0000
CONDEMNING LIMIT OF RAIL HEAD (FRACTION OF RAIL HEAD)            0.2500
GRADIENT OF TRACK (PERCENTAGE)                                   0.0000
CORROSION CONSTANT FOR CURVE(INCHES SQUARED/YEAR)              0.005200
TANGENT WEAR RATE (INCHES SQUARED/MGT)                         0.000560
INCREMENTAL CURVE HIGH RAIL WEAR RATE(INCHES SQUARED/MGT)      0.000715
CURVE SEGMENT FILE                                           SCURVE.DAT


WHEEL LOADS (KIPS)    PERCENTAGE
     10.0000            25.000
     20.0000            35.000
     30.0000            35.000
     40.0000             5.000
*******************************************************************
```

MP (FROM)	MP (TO)	CURVATURE DEGREE	HEAD WEAR RATE (IN**2/YR.)	RAIL LIFE 119 LBS/YD YEARS	(MGT)
1.000	2.000	0.5000	*	*	*
2.000	3.000	1.0000	*	*	*
3.000	4.000	1.5000	*	*	*
4.000	5.000	2.0000	0.1180	9.15	274.64
5.000	6.000	2.5000	0.1417	7.62	228.59
6.000	7.000	3.0000	0.1646	6.56	196.82
7.000	8.000	3.5000	0.1865	5.79	173.69
8.000	9.000	4.0000	0.2075	5.20	156.13
9.000	10.000	4.5000	0.2277	4.74	142.32
10.000	11.000	5.0000	0.2471	4.37	131.14
11.000	12.000	5.5000	0.2723	3.97	118.97
12.000	13.000	6.0000	0.2977	3.63	108.84
13.000	14.000	6.5000	0.3231	3.34	100.28
14.000	15.000	7.0000	0.3485	3.10	92.96
15.000	16.000	7.5000	0.3740	2.89	86.64
16.000	17.000	8.0000	0.3994	2.70	81.11
17.000	18.000	8.5000	0.4249	2.54	76.25
18.000	19.000	9.0000	0.4504	2.40	71.93
19.000	20.000	9.5000	0.4759	2.27	68.07
20.000	21.000	10.0000	0.5015	2.15	64.60
21.000	22.000	10.5000	*	*	*

Construction of the Tumbler Ridge Branch Line in Northeastern British Columbia

V. W. Shtenko*

1. PROJECT OUTLINE

In 1976, the British Columbia Government funded a study (Phase I) to conduct preliminary engineering to access the Northeast Coal Block by rail. The British Columbia Railway was requested to carry out the rail feasibility study.

Two main routes were studied, one originating at Chetwynd and running in a southeasterly direction and the other originating at Anzac and running in an east-northeast direction. Both Chetwynd and Anzac are located on the B.C. Railway mainline, north of Prince George. Construction costs were determined using mapping developed to a scale of 1:5000. These costs plus operating and maintenance costs over a given duration were placed into a computer model to determine the most desirable route based on the costs developed and operating parameters.

The result of the study was that a route studied by C.N. Rail (Monkman I) was most desirable, with the B.C. Railway route, Anzac-Tumbler Ridge, being a very close second. Monkman I followed the McGregor River Valley and was in conflict with the B.C. Hydro plan to divert the McGregor River and flood the valley; the Provincial Government chose the Anzac route (Figure 1) as it wished to retain the McGregor River diversion option.

In 1977, B.C. Railway was requested to study the Anzac route in greater detail to obtain a more accurate and current estimate of construction costs (Phase II). Mapping was developed to a scale of 1:2000 using photography with good vertical and horizontal ground control. Based on the new mapping data, terrain analysis and limited field information, an estimate of costs was developed. The branch line, 105 km in length, was estimated to cost $189 million including three tunnels, each being approximately 5 km in length. This portion of the study was completed in 1978. In 1980, this study was updated to include the extension of the branch line from 105 km to 130 km. The resultant costs were estimated to be $310 million in 1980 dollars which included $30 million for the extension of the branch line.

With negotiations in mid-1980 progressing favourably, the Provincial Government approved the conduct of detailed investigations and designs of the proposed major tunnels, including cost estimates, followed by approval to proceed with the construction of the access roads and preparation of documents to construct the first 30 kilometres of the grade.

Project approval was received in December, 1980, which was followed by mobilization of engineering forces to conclude the design and supervise the construction of grade, bridges, tunnels, and track. A budget in April 1981 dollars of $374 million was developed, which finally was adjusted to an "in-place" budget of $455 million (which included electrification).

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NOTE: Due to requirements of bulletin format some of the material in the figures in this article were reduced in size to the extent that some of the material in the figures might not be readable. Those wishing more detail information can obtain this by requesting the information from A.R.E.A. headquarters by January 10, 1985 for the price of 25¢ per sheet plus $1.00 for postage and handling. Please specify the page number of which you want enlargements.

Design activity commenced immediately involving B.C. Railway personnel and consultive services covering all aspects of the branch line. Grade design generally commenced from Km 0 and sequentially progressed to Km 130. Construction progressed in a similar fashion. Design of the two major tunnels (Table and Wolverine) commenced with the detailed exploration program launched in late 1980 and was concluded in April, 1981. Following detailed investigations, a decision was made to combine Tunnel Nos. 1 and 2 into a single 9 km tunnel, to be known as Table Tunnel. The former Tunnel No. 3 was renamed as Wolverine Tunnel. Tenders for the tunnels were opened on September 25, 1981 and four contracts awarded on December 9, 1981. Similarly, bridge designs commenced in early 1981 and were concluded in mid-1981. With respect to the two major tunnels, the original concept of providing ventilation and a system of doors (to provide cooling air for the locomotives) was rejected in favour of electrifying the branch line using electric locomotives on a 50 kV system.

The grade and all bridge structures were completed by September, 1983, while the Wolverine and Table Tunnels were "holed through" on May 28, 1983 and August 31, 1983, respectively. The two short tunnels at Km 80 and Km 86 were completed by July, 1983.

The design of the electrification system commenced in mid 1982, with installation contracts awarded in July, 1983. Installation was concluded by December 15, 1983 in the Wolverine River Valley between Km 77 and Km 130 (including the loadout loops) and between Km 54 and Km 24 in the Table River Valley. However, the installation of the overhead contact system between Km 0 and Km 24 was rescheduled for early 1984 (the poles were installed in 1983). Energization of the line commenced in late November, 1983 and was concluded in February, 1984.

With respect to trackwork, in particular in consideration of placing CWR vs jointed rail, based on economics and possible late delivery of one of the tunnels, it was decided to first lay 78 foot existing new and used jointed rail, followed a year later with a relay using continuous welded rail. Further, it was decided that all track material required (rail, ties, spikes, plates, etc.) between Km 70 and Km 130 would be transported from Chetwynd to the Wolverine River Valley and stockpiled in the vicinity of Km 100. This operation commenced in January, 1983 and was completed by mid March, 1983. Rail laying and ballasting commenced in June, 1983, and was concluded on October 21, 1983.

The first coal train departed the mine site on November 1, 1983 to tidewater at Prince Rupert, one month ahead of schedule.

2. INTRODUCTION

In the negotiations carried out in 1980 between the prospective coal producers, Government officials, and Japanese authorities, the date to commence coal shipment was set at December 1, 1983. To achieve this, the installation of the infrastructure was also to be completed by December 1, 1983. This consisted of the development of a new townsite of Tumbler Ridge, development of the mine sites and processing plants, construction of a highway between Chetwynd and Tumbler Ridge, construction of the railway between Anzac and Tumbler Ridge, and the development of port facilities at Prince Rupert.

Since the negotiations were progressing favourably, by mid July, 1980, and upon recommendation by the British Columbia Railway, the Government of British Columbia decided to commit funds to carry out essential engineering designs and development of contract documents in order to keep the project on the critical path and still meet the completion date of December 1, 1983. The items that were considered to be critical were: investigations and design of the tunnels, construction of all access roads along the Table, Sukunka and Wolverine Valleys, soil investigations and design of the grade from Km 0 to Km 30.

By September, 1980, consultants were engaged to carry out the necessary field investigations and final designs for the tunnels which would be used for preparation of contract documents and cost estimates. The field drilling and mapping programs were concluded in December, 1980. Designs, contract-type cost estimates and final reports were concluded in April, 1981.

During tunnel field investigations, it became apparent that the depth of overburden at the east portal of Tunnel No. 1 would be substantial (greater than 50 metres). It was decided, therefore, to combine Tunnels No. 1 and No. 2 into one long tunnel, approximately 9 km in length. This tunnel would connect the Table and Sukunka Valleys, and would be known as the "Table Tunnel". All other portals were confirmed, generally in the same locations as recommended in the 1977 - 1978 study (Phase II). It was also decided to refer to Tunnel No. 3 as the "Wolverine Tunnel".

Field investigations and surveys for the first 30 km of grade commenced in the fall of 1980 and was concluded in the spring of 1981 which included detailed design and preparation of contract documents.

With respect to the access roads, contract documents were prepared in December, 1980, using all available data, and only field reconnaissance surveys. Forest Service standards were adopted in the design of the access roads. Contracts were awarded in January, 1981 to construct 27 km of access road in the Table River Valley and 30 km in the Sukunka River Valley which would provide access to the east and west portals of the Table Tunnel and west portal of the Wolverine Tunnel. These access roads would be extensions to existing Forestry roads. Construction also included the construction of a number of bridges. Completion of access road construction was scheduled for August 15, 1981. The Wolverine access road, approximately 20 km in length, was less difficult, and preparation of contract documents and construction was scheduled for mid 1981. Reference should be made to Figure 2 which shows the Tumbler Ridge Branch Line in plan indicating significant features.

Agreement in principle was reached in mid December, 1980 between the Government, coal producers, and Japanese interests. With this agreement, the B.C. Railway Engineering Department was requested to form a Project Management Team (task force) and proceed with all phases of design, contract document preparation, and construction. The Project Management Team was formed utilizing existing B.C. Railway Engineering staff and supplemented by consultive services and new employees.

With the data developed in previous studies and information obtained such as geotechnical investigations and design for the first 30 km of the grade, tender prices to construct the Table and Sukunka access roads, tunnel investigations, design and cost estimates, a budget was developed in April, 1981 dollars. The April, 1981 budget was further adjusted by applying a 1% inflation rate per month on the schedule of planned expenditures over the life of the project, giving an "in-place" budget (see Table 1). In developing these costs, the grade and track standards used previously were applied with the exception that the weight of rail was reduced from 136 lbs. to 115 lbs. which was based on annual traffic projections of 8.5 million net tonnes, ± 10%.

TABLE 1

	April, 1981 Budget	"In-place" Budget	"Present In-place" Budget
Access Roads	$ 9,265,000	$ 9,738,000	$ 9,738,000
Grade	$ 88,111,000	$103,413,000	$103,413,000
Tunnels	$152,499,000	$183,616,000	$165,463,000
Bridges	$ 17,422,000	$ 20,559,000	$ 20,559,000
Track	$ 34,979,000	$ 45,936,000	$ 45,936,000
Electrification	-	-	$ 20,863,000
Infrastructure	$ 10,687,000	$ 12,087,000	$ 12,087,000
Other	$ 3,785,000	$ 4,437,000	$ 5,525,000
Engineering	$ 28,501,000	$ 33,055,000	$ 34,386,000
Project Reserve	$ 28,549,000	$ 34,146,000	$ 36,705,000
Total Project	$373,798,000	$446,987,000	$454,675,000

NOTE: The budgets shown are arithmetic summation of each segment for each category, which tends to indicate an accuracy of ± $1,000. Further, in June, 1982 a decision was made to electrify the Branch Line which permitted the elimination of tunnel ventilation. This change in scope led to the change in the "Present In-place" budget, as shown.

3. PROJECT MANAGEMENT TEAM

A project team was established at very short notice within B.C.R's Engineering Department to manage the development of the project and complete it within three years.

A number of factors affected the structure of the organization. Because of the large volume of work that had to be completed in less than three years, a fast acting, responsive structure was needed. The construction site was located in an isolated area and was physically divided in half by two mountain ranges. Communications and transport within the area and with the outside world were limited and difficult. It was necessary to construct and operate camps and other infrastructure for project staff. The lack of lead time meant that project management systems and procedures had to be developed as the project progressed.

The organization structure also reflects the need for active participation by the owner in the management of the project. Thus, a project team was assembled in-house to be responsible for implementing overall policies, guidelines, and design concepts as well as controlling project progress and taking corrective action.

Overall project management was the responsibility of the Project Manager, who was also the railway's Chief Engineer. Under him, the responsibility was divided into three functional groups -- design, construction supervision, and administration, as shown in Figures 3 and 4.

The design group, located in B.C.R's North Vancouver head office, was responsible for coordinating a multi-discipline team within B.C.R.'s engineering department. The group covered bridge design, track, geotechnical structures, line location and other facilities; its responsibilities included preparation of contract drawings and specifications.

In some cases consultants were retained to carry out specific design assignments. In particular, consultants were responsible for the design of the major tunnels and the electrification system.

To expedite construction, as soon as design of one section of the formation was completed, contracts were prepared and let. Meanwhile, surveys, soil investigation, and design work continued for successive sections of formation and bridges further along the line. This practice was followed progressively from one end of the line to the other and a series of contracts was awarded during 1981 and 1982.

The construction supervision group was responsible for supervising the work of the contractors, administering the contracts on site, maintaining quality control, and coordinating the work. Owing to the physical constraints of the construction site, the construction manager was located in Prince George, a major centre on the rail system, with microwave communications links to both the head office and the worksite, while the site staff was divided into three groups, each under a project engineer.

Two of the project engineers were responsible for all work being done on one side or the other of the mountain. The third was in charge of tunnel construction. Each project engineer was assigned several resident engineers who were charged with supervising the work of one or more contracts. They were assisted by a group of inspectors and surveyors.

The administration group was located in B.C.R.'s head office in North Vancouver and was responsible for providing project management services. This required the development of systems and procedures for cost accounting and reporting, contract formation, planning, scheduling and reporting, materials procurement, and computer applications to monitor progress on the project and provide control information.

The planning and scheduling system, based on critical path techniques, was developed to provide details on completion dates, cost estimates and project outlooks. A standard commercial software package was adapted for this application.

The cost accounting package, which complemented the scheduling information, was developed to report actual costs, or estimates of these costs, as they were incurred, on a monthly basis. The quantity calculation system was developed to calculate earth and rock excavation volumes, based on ground surveys and design details, and using stand-alone microcomputers in remote locations.

A three member advisory board was assembled, consisting of recognized experts in the fields of geotechnology, tunnelling, and railway design and construction. Reporting to the project manager, they reviewed all engineering work and provided a second opinion on its quality and adequacy.

Environmental concerns and the interface with other authorities was the responsibility of the Environmental Coordinator located in Prince George. Although the various agencies dealt directly with contractors and others in issuing and enforcing permits and licences, his effort was spent in liaising with the agencies and the contractors to expedite the work while meeting the requirements of the regulations.

As far as possible, work was carried out using in-house services. All managerial and supervisory positions were staffed by B.C.R. employees. Where staffing levels were insufficient to cope with the volume of work, additional personnel were hired. Extensive use was made of the materials management, communications, data processing, industrial relations, mechanical, and finance departments within B.C.R.

4. ACCESS ROADS

Construction of the Table River and Sukunka River access roads commenced in January, 1981 and were concluded in the fall of 1981. On account of poor weather conditions, poorly drained soils and inability of the contractor to react positively to conditions encountered, the scheduled completion of the Sukunka River access road was not achieved until late fall, 1981. Construction of the Wolverine River access road commenced in July and was concluded on schedule. The Table River access road was completed on schedule, although surfacing (placing granular material) continued into late fall. The access roads are all weather roads, although maintenance requirements were extensive in certain sections.

The access roads constructed by British Columbia Railway for the Tumbler Ridge Branch line are extensions to existing Forestry roads. These new roads could be used by the Ministry of Forests to open up new areas for logging in the future. Certain sections of these roads, generally in the vicinity of the tunnel portals, have been designed as required only for construction and consequently, once construction is concluded, stream crossings will be removed and thus revert to nature.

5. GRADE

5.1 Design

Final grade design was a refinement of the original preliminary design proposed in Phase II (1977 study), but also incorporating the additional information obtained from the field surveys, geotechnical programs, seismic surveys, hydrological studies, and avalanche monitoring. The following paragraphs briefly describe: 1) Surveys, 2) Geotechnical Investigations, 3) Hydrology, 4) Avalanche Monitoring, 5) Environmental Considerations, and 6) Alignment and Quantity Calculations.

1) Surveys - To prepare contract documents, B.C. Rail mobilized survey crews in the field. These field surveys provided control for construction and established accurate datum from which grade design and construction plans were prepared. Field surveys proceeded as follows:

   - A control survey was run along the route and control points established near the proposed centre line. This survey was tied into the control survey which was used in Phase II for aerial photography and mapping and which is based on the U.T.M. system with geodetic elevations. A centre line survey was then run, based upon the preliminary route design. Bench marks and reference points were established at regular intervals for use later during construction. Centre line profile and cross sections were then taken to provide a topographic base. All geotechnical test holes and hydrological features were tied into this survey.

2) Geotechnical Investigations - A geotechnical field investigation program was undertaken to bring the data to an acceptable level to prepare tender documents. The program consisted of three phases of field work which included; drilling, laboratory testing, supplementary field drilling and exploration, and finally the implementation of the results for the design of the alignment, grade and cross section templates.

   Prior to starting the field drilling and exploration phase, information obtained during field reconnaissance and terrain analysis, and landform identification carried out during Phases I and II (pre-engineering studies carried out in 1976 and 1977, respectively) was used to lay out and specify the depth and location of test holes. A complete drilling (including test pits) program was developed.

   The first phase of the geotechnical program included all drilling, mapping of landforms, test pits and visual reconnaissance necessary for the design of the roadbed. This field work was carried out by contractors and consultants working under the supervision of B.C. Railway Engineering staff and included verification of soil types identified in the terrain analysis which was completed in Phase II (1977 study), determining soil strength parameters, assessing the suitability of soils for use as fills, identifying the sources of granular material for sub-ballast, identifying potentially unstable landforms, and establishing stripping depths for embankments and subcuts in excavation areas. The drilling program consisted of using a track mounted power auger. Test pits were excavated

using backhoes. The locations of these drill holes and pits were laid out to assist in soil identification and borrow pit investigations. This work was supervised by B.C. Rail technicians. Access for the work was marginal in many areas. All terrain vehicles and helicopter support was required at times. Visual inspections and observations formed part of the program.

The second phase of the Geotechnical program consisted in the systematic laboratory testing of recovered soil samples and test hole log development to include in the final drawings. Tests included visual laboratory classification, moisture content and Atterberg limits, unconfined compression, triaxial compression, direct shear, consolidation, sieve analysis, and maximum density (Proctor) tests. Summarized test hole logs were drafted onto terrain analysis sheets and abbreviated logs were shown on the plan profile sheet.

Once this data had been assimilated into the design, any design changes would require field verification. For this purpose, supplementary field drilling and exploration was carried out as construction proceeded.

The third and final phase of the geotechnical program was to utilize all the geotechnical information to finalize the geometric design of the roadbed and provide specialized designs in problem areas. These special designs included retaining walls, subdrain systems, benches and berms, slides and instabilities, surcharge areas, sub-cut areas, and areas where instrumentation would be required.

To complement the drilling program and subsequent testing, a survey of the proposed centre line was made using seismic refraction. This was done to assist in establishing a probable rock profile for the route. The results of this work augmented the field drilling program. Actual test drill holes were utilized in calibrating the seismic results.

3) Hydrology - Specialists were retained to prepare a detailed hydrological survey to determine the final design flows for bridge and culvert locations. Stream crossings were examined in the field to confirm the estimates of stream flows which were made based upon topographic maps and existing rainfall data. Field estimates were made based upon actual stream dimensions, visible signs of high water, slope and characteristics of stream bed, and any other factors which may aid in the assessment of actual flow. The consultants' final report was used as the basis for all bridge and culvert design and for drainage along the grade.

The hydrological criteria applied to design flows were as follows:

1) Bridges - 1 in 200 yrs.

2) Culverts - 1800 mm in diameter or greater - 1 in 100 yrs.

3) All other - 1 in 50 yrs.

4) <u>Avalanche Monitoring</u> - Avalanche conditions along the proposed alignment have been monitored annually and reports submitted since 1978. These reports have been prepared through the assistance of experts from the Federal Government.

Once construction on the line was started, B.C. Rail retained an Environmental Coordinator to develop an avalanche and snow monitoring program. This would highlight danger areas during construction and future operations.

The field program initiated by B.C. Rail in 1981 and which will continue into operating years consists of the following:

- Daily weather observations.

- Records of avalanche activity (natural and artificially released).

- Stabilization (explosives) of unstable slopes and release of potentially dangerous cornices.

- Warning signs for access roads.

- Comprehensive search and rescue program.

- Standard first and second party rescue packs at the Sukunka and Whitford Camps where avalanche conditions are most likely to occur.

The weather and snowpack sections of the program are based upon the NRC Technical Memorandum No. 132 - Guidelines for Weather, Snowpack and Avalanche Observations. The search and rescue sections were based largely upon existing M.O.T.H. plans and discussion with M.O.T.H. personnel.

The avalanche studies and observations which have been completed and which are continuing have resulted in constructing the following facilities to protect against avalanche activity:

- At Km 42.2 and Km 47.3, the grade is constructed as a fill to a minimum height of 15 metres to minimize avalanche impact.

- At Km 42.4, Km 42.6, Km 43.4, Km 48.6, Km 49 and Km 41.2 to Km 42, earth deflectors are planned to be constructed to divert and reduce the energy of avalanches.

- Throughout all avalanche areas "through cuts" have been daylighted on the downhill side to allow passage of snow avalanches and to provide for easy clean up and removal of debris.

5) <u>Environmental Considerations</u> - The British Columbia Ministry of Industry and Small Business Development retained consultants to carry out a study and prepare a Stage Two report on the environmental impact of the Tumbler Ridge Branch Line. This report would also cover the Hook Lake Bypass alternative. The report was concluded by November, 1980.

The report considered the impact on the following resources: Forestry, Wildlife, Fisheries, Agriculture, Mineral and Petroleum, Heritage, Social Impacts, and Land Status. The report concluded that the impact of the Branch Line would not be severe. Recommendations were made to minimize the effects of construction. Guidelines were set for rehabilitation upon completion of construction.

6) Alignment and Quantity Calculations - Final centerline locations and profiles were established from field surveys and 1:2000 scale mapping. Tentative alignments and profiles were plotted and quantities calculated. This process was repeated with progressive adjustments until an acceptable alignment and balance of quantities had been established. In some cases, due to the nature of available materials and topography, a balance of materials was not always desirable. In this case, borrow areas were designated to provide for these discrepancies in the materials along the right-of-way.

Geotechnical evaluation proceeded simultaneously with the location using data from field test holes, seismic refraction and terrain analysis. This evaluation was used to determine roadway side slopes, stripping requirements, waste of unsuitable materials, borrow pits, etc.

Earthwork quantities were computed by applying the design cross sections to the surveyed ground line. Volume quantities were calculated by the "Average End Area" method. Calculations were performed by B.C.R. personnel on British Columbia Systems Corporation's (BCSC) IBM 3033 using their Metric Design System (MDS).

In using this comprehensive design package, the optimization of the center line location and quantity determination was done quickly and efficiently with many options reviewed. The program was able to accept original ground sections, bedrock location, material usage and alignment data. With the entering of the design templates and alternate alignment proposals, the program automatically recalculated the alignment and quantities producing a variety of reports and plots, (i.e. OG listing, SR listing, quantity listing, overhaul calculation mass haul diagram, X-section plots, 3-D perspectives and plan profile plots).

Quantities were then summarized into a design file which was subsequently used by the Resident Engineer to monitor the project and for tender quantities in the contract.

Figure 5 represents a typical plan and profile that was provided with the Contract Documents. The details provided consist of: 1) topography, 2) railway alignment and right-of-way limits, 3) drill hole locations, 4) terrain units, 5) center line profile, 6) grade line, 7) simplified drill logs, 8) cut and fill designs, 9) drainage requirements, 10) environmental concerns, 11) borrow and waste areas, etc. In addition to the plan and profile, a plan showing the terrain analysis and detailed drill logs are also included with the documents.

Reference should be made to Table 2 which shows the planned (P) schedule and actual (A) for the design and tendering of the railway grade.

5.2 Construction

The construction of the railway grade, which includes placement of sub-ballast, was divided into a number of sections. This was done to attract medium sized construction companies, and to accommodate the various types of terrain. The sections are as follow: Km 0 - Km 26, Km 26 - Km 42, Km 42 - Km 55, Km 64 - Km 71, Km 79 - Km 97, Km 97 - Km 116, Km 116 - Km 121, and Km 121 - Km 130. The portion between Km 116 and Km 121 is being constructed by the Department of Highways, as the railway and highway occupy a common right-of-way in this area. Table 2 shows the planned and actual construction activity.

The following provides a brief description of the design and construction features on each segment of the grade:

1) Km 0 - Km 26

This portion of the grade follows the Parsnip River and lower reaches of the Table River. The materials encountered consist of gravels and sands, wet clayey silts, ad normally consolidated alluvial materials. The fill slopes varied from 1-1/2:1 to 2:1, while the cut slopes were 2:1 to 2-1/2:1, with benches at some locations. Some fill areas required subexcavations, while others were traversed utilizing geotextile fabrics although the subgrade widths were increased to accommodate the anticipated settlements.

Construction commenced in July, 1981 and was substantially concluded in October, 1982. Placement of sub-ballast and minor cleanup was concluded in 1983.

2) Km 26 - Km 42

This section of the grade is generally a sidehill location along the Table River. The materials were similar to that encountered between Km 0 and Km 26, with only a limited amount of bedrock being encountered. Construction commenced in September, 1981 and was substantially completed by November, 1982. Placement of sub-ballast and minor cleanup was concluded in 1983.

3) Km 42 - Km 55

This portion of the grade follows a sidehill location along the Table River, although a large portion is founded in bedrock. The overburden depths are fairly variable and are generally of a high water content. The most westerly sections (Km 52 - Km 55) consist of soils which are largely fine sands and silts. This section also contains a number of avalanche areas which would not require snowsheds, but may require the construction of some type of deflecting structure. Construction commenced in March, 1982 and was completed in September, 1983.

4) Km 67 - Km 71

This section of the grade is within the Sukunka River Valley and the soils are generally wet and normally consolidated. The foundation is prepared by subexcavation or placement of geotextile fabric. Construction commenced in July 1982, which consisted of clearing, subexcavation and placement of fill material derived from the tunnel excavations. The grade in this area will generally be a tunnel muck fill of at least two metres in height and was completed in October, 1983.

5) Km 78 - Km 97

The grade follows a sidehill location along the Wolverine River. The material is a combination of bedrock and glacial tills being predominantly silty sands and gravels. Construction commenced in October, 1981, and was completed in July, 1983, including placement of sub-ballast.

6) Km 97 - Km 116

The grade is generally located on the Wolverine River floodplain with periodic sidehill locations. The materials encountered are predominantly dry silty sands and gravels with some very wet normally consolidated fine grained soils and some wet unstable sidehills. Generally speaking, the wet normally consolidated areas are traversed by fills and as such, geotextile fabric is utilized, while the unstable sidehills are drained and constructed to a cut slope determined through a stability analysis.

Construction commenced in June, 1982, and was substantially completed in Decmeber, 1982, which included the construction of the Teck load-out loop (approximately Km 115). Sub-ballast was placed in 1983.

7) Km 116 - Km 121

Work on this section began in August, 1982 and was completed in August, 1983. This section is being constructed by contractors who are supervised by the B.C. Ministry of Highways. The railway grade runs parallel with a new highway (common right-of-way). A joint contract was let for the construction of highway and railway grade.

8) Km 121 - Km 130

This section of the grade follows the Murray River and is a sidehill location for the first 7 km, with the remainder being along the uplands. The soils are generally dry silty sands and gravels with isolated bedrock. Construction on this section began in June, 1982, and was completed by July, 1983, being 2 weeks ahead of schedule.

6. BRIDGES

The Tumbler Ridge Branch Line consists of eleven (11) bridges, with six (6) being in the Table River section, and five (5) in the Wolverine River section. Phase II (1977 study) provided the base data utilized in preparing the final designs for all bridge structures including the type of superstructure and substructure for each bridge. The designs for the larger structures such as the Parsnip River Bridge, Boulder Creek Bridge, and Km 80 Bridge, were carried out by consultants, while the smaller structures such as Tacheeda Creek and Bullmoose River Bridges, which generally are pre-stressed concrete box girder spans, were designed by B.C. Railway personnel.

The designs commenced in February of 1981, starting with the Parsnip River, Table River, and Boulder Creek Bridges, and concluded in May, 1982 with the Bridges in the Wolverine Valley. Table 3 shows the design and construction schedule for all the major bridges relative to original plan (P) and actual (A).

The bridge structures on the Branch Line can be categorized into four basic types of structures, namely 1) pre-stressed concrete box girder spans, 2) steel deck plate girder spans, 3) steel through truss spans, and 4) continuous steel box girder spans with steel delta legs supported on concrete piers. The four types of structures described above are represented by Figures 6 to 9, respectively. The decks will be either reinforced concrete or steel troughs, which will accommodate the ballast and track structure. The substructures consist of concrete abutments, concrete piers, steel "H" pile piers with concrete caps, and steel "H" towers or just steel towers.

The following provides a brief summary description of each bridge on the Branch Line:

1) Tacheeda Creek Bridge (Km 1.0)

This structure consists of three 12.2 metre pre-stressed concrete box girder (ballasted deck) spans founded on concrete abutments and two concrete capped steel "H" pile piers. The substructure was concluded in August, 1982, with the concrete spans placed into position by October, 1982.

2) Parsnip River Bridge (Km 3.0)

This bridge is one of the 6 major structures consisting of two 84 metre steel through truss spans. The superstructure rests on two concrete abutments and one mid-stream concrete pier. A concrete ballasted deck is provided with a metal walkway. Construction commenced in August, 1981 and was concluded by September, 1982. The superstructure was assembled on the adjacent grade and launched into position.

3) Wooyadilinka Creek Bridge (Km 4.0)

This structure is similar to the Tacheeda Creek Bridge which consists of three pre-stressed concrete box girder spans, one being 13.7 metres and the other two being 10.7 metres in length. The superstructure is set on concrete abutments and two concrete capped steel "H" piles. Construction commenced in July, 1982 and was concluded in October, 1982.

4) Crocker Creek Bridge (Km 24.3)

The superstructure consists of four 13.7 metre pre-stressed concrete box girder spans. This structure is also supported on concrete abutments and three concrete capped steel "H" pile piers. Construction commenced in July, 1982 and was concluded in October, 1982.

5) Table River Bridge (Km 28.7)

This structure consists of three 26.1 metre steel deck plate girder spans, supported by concrete abutments and two concrete piers. The deck consists of reinforced concrete which will accommodate the ballast and track structure. Construction commenced in August, 1981 and was concluded in June, 1982.

6) Boulder Creek Bridge (Km 32.8)

The superstructure consists of four steel deck plate girder spans resting on three steel towers and concrete abutments. Two of the spans are 26 metres, while the others are 30 metres and 16.5 metres in length. Construction commenced in August, 1981 and was concluded in September, 1982.

7) Lost Creek Bridge (Km 80.1)

The structure consists of one 61 metre steel deck truss span with two (one 13.6 metres and the other 18.6 metres) steel deck plate girder approach spans. The foundations consist of concrete piers and abutments. The deck will be reinforced concrete to accommodate the ballast and track structure. Construction commenced in July, 1982 and was concluded in July, 1983.

8) Two Creek Bridge (Km 110)

This creek is crossed using one 15.8 metre and two 12.2 metre pre-stressed concrete box girder spans on concrete abutments and two concrete capped steel "H" pile piers. Construction commenced in July, 1982, with the substructure being completed by December, 1982. The placement of the superstructure was concluded by July, 1983.

9) Bullmoose Creek (Km 116.7)

This bridge consists of six 15.85 metre pre-stressed concrete box girder spans on concrete abutments and five steel "H" tower structures capped with metal grillages. The substructure foundation was completed in 1982, and was concluded by July, 1983.

10) Wolverine River Bridge (Km 118.1)

This structure is a continuous steel box girder with two delta legs supported on two concrete piers and concrete abutments. The total length of the bridge is 132 metres, with the deck being a steel trough which forms part of the girder structure and which will accommodate the ballast and track structure. Construction commenced in August, 1982 with the substructure completed in December, 1982, and with the superstructure being assembled on site and launched into place in March, 1983. The structure was completed by August, 1983.

11) Murray River Bridge (Km 120.7)

This bridge is similar in design and construction to the Wolverine River Bridge except it will contain three delta legs supported on concrete abutments and three concrete piers, with one pier being located in the river. The length of this bridge is 157 metres. Construction commenced in August, 1982, with the substructure being completed in December, 1982, with the superstructure being assembled on site and launched into place by May, 1983. This bridge was completed by mid August, 1983.

7. TUNNELS

In September, 1980 the tunnel consultants were authorized to proceed with the geological investigation, designs, and provide a contractor type of cost estimate covering Tunnels #1, #2, and #3 (as referred to in the Phase II study). The scope of the work included:

(a) Geological exploration of the study areas, including mapping, drilling and geophysical surveys.

(b) Topographic surveys of portal areas.

(c) An evaluation of geology along tunnel alignments.

(d) Selection of tunnel alignments, grades and portal locations.

(e) A prediction of tunnelling conditions, as dictated by rock quality and geological hazards such as fault zones and groundwater.

(f) Tunnel design, including permanent support requirements, drainage and ventilation.

(g) Construction considerations, including camp locations, construction adits, muck disposal, aggregate sources and contract packages.

(h) Contractor type of cost estimate.

Upon commencement of the field work, the original investigations program was modified to concentrate on what was considered the more essential elements of the field program, which were: geological mapping and drilling to confirm earlier judgements, particularly with regard to portal locations; the extent of possible fault zones the tunnel would cross; and groundwater conditions to be encountered during tunnelling.

During the exploration program performed in late 1980, it was found that the overburden at the proposed east portal of Tunnel #1 (Figure 1) was deeper than first anticipated which would involve substantial excavations to be carried out. Also, access to the east portal of Tunnel #1 and west portal of Tunnel #2 was somewhat difficult and would have extended the already major undertaking of providing access to tunnel portals in the Table River and Sukunka River Valleys. The decision was made to combine Tunnels #1 and #2, to be known as the Table Tunnel, and thus eliminating the two portals and minimizing the number of portal facilities that would be required for the ventilation systems.

With respect to Tunnel #3 (Figure 1), renamed the Wolverine Tunnel, the investigations carried out generally confirmed the alignment, conditions to be encountered at the portals, and portal locations as identified in the Phase II study (1977).

The field investigations were concluded in November, 1980, with the final report and cost estimate submitted in April, 1981. The results of these investigations, and subsequent analysis are represented on Figures 10 and 11, showing the geological profile of the Table and Wolverine Tunnels, which highlights such features as: principal rock

type, rock quality classification, anticipated groundwater flow, fault zones, proposed type of tunnel support, etc. Typical cross sections are shown on Figures 12 and 13 which detail the three types of support proposed, namely Type I, Type II, and Type III.

Based on the cost estimates that were developed, it became apparent that the Wolverine Tunnel unit cost (cost per metre) would be more expensive than for the Table Tunnel. This led to consideration of bypassing the Wolverine Tunnel, which has been referred to as the Hook Lake Bypass and is briefly described below:

> "When the Railway decided to locate the rail line along the Table River Valley, it was committed to construct the Table Tunnel since there is no other way of traversing between the Upper Table Valley and the Sukunka, and maintaining the ruling gradient of 1.5%. This is not the case for the Wolverine Tunnel. There is an alternative overland route (Hook Lake Bypass) which would proceed east along the Sukunka Valley to Hook Lake, then northwest into the Upper Wolverine Valley (see Figure 2). This route had been considered previously and had been rejected on the basis of avalanche conditions, very rugged topography, and the additional 12 km of length involved with the route.
>
> The drilling and exploration work carried out in 1980 indicated that the rock conditions for the Wolverine Tunnel were not expected to be as good as those for the Table Tunnel. This led to a further look at the Hook Lake Bypass as an alternative to the Wolverine Tunnel. A survey was made of the route on the ground with the same conclusions as established at first: that the route would be difficult and expensive, but that it was a viable alternative. This was important at the time. In the event the tenders for the Wolverine Tunnel were unreasonable due to the predicted poorer ground, the Company would have the option of going the Hook Lake Bypass route. This option did not materialize, since tenders and projected cost to completion for both tunnels were within reasonable budgetary limits."

Members of B.C. Rail staff, along with personnel from the consultant, made a field visit to Flathead Tunnel in Montana, U.S.A. in January, 1981. This tunnel is 11 kilometres long and is operated by Burlington Northern Railway. It was anticipated that the ventilation systems for the Tumbler Ridge Branch Line would follow the general arrangement that exists at Flathead.

The Moffat Tunnel was also visited to observe operating conditions. The principle of the above ventilation systems is that a door and fan system is provided at one end, usually the upper end, of the tunnel. When the train enters the lower end of the tunnel, the door is closed. As the train moves toward the door, a piston effect is created which forces air past the locomotives and cools the engines. Air passing the locomotives assures clean air for the crews and engine intakes. As the train approaches the door, it opens and the train leaves. The door then closes and the fans start up to purge the tunnel of hot air and fumes left by the train.

While the above systems function adequately, they are a continuing maintenance and operating problem, and more importantly, a bottleneck to increasing traffic volume.

Design concepts which would provide adequate ventilation to satisfy existing B.C. Workers' Compensation Board regulations are not totally understood and are still somewhat empirical. Capital and operating costs to construct and operate a system that would probably satisfy compared to one that would certainly satisfy the above regulations could be considerable. The alternative to ventilation is to operate electric trains through the tunnels. In May, 1982, British Columbia Railway made the decision to electrify the Branch Line and eliminate the ventilation requirements.

The design and contract documents for construction of the tunnels were developed and prepared, based upon the results of the field exploration program which included drilling seismic surveys, petrographic analysis, permeability analysis and geological mapping. Tenders were received on September 25, 1981 from seven contractors to drive the Table and Wolverine Tunnels. Four contracts were awarded on December 9, 1981 for construction to proceed from four headings. It should be noted that all bidders were pre-qualified in June, 1981, followed by a site visit and briefing, all prior to preparation of bids.

Contracts for the excavation of portal overburden and portal preparation were awarded and work commenced by mid 1981, and concluded in the winter of 1982. Three camps to be occupied by the tunnel contractors were also installed during the summer of 1981. These camps were provided to the contractor with a capacity of 50% of the anticipated manpower required to drive the tunnels. It was the tunnel contractors' responsibility to expand the camps to satisfy their total requirements. The three camps were located as close to the tunnel work as topography and soil conditions would allow. The camps were constructed before the tunnel contracts were tendered. This provided for the tunnel contractors to mobilize immediately following the award of contracts.

The contracts having been awarded on December 9, 1981, the contractors mobilized with the intention of driving tunnel by April, 1982. Equipment was purchased and fabricated and moved to the site. All headings were driven by conventional methods of drilling and blasting. This was done using hydraulic drill jumbos with six drills mounted on each jumbo. The drill jumbos operated on a sliding floor. Mucking was by use of Conway and Hagglund loaders, and the muck was transported by track mounted cars.

Tunnel excavation commenced in April, 1982 as planned. The more difficult problem turned out to be the portal excavations and the preparation of the portal face to an acceptable standard for tunnelling. The tunnels were completed as follows:

- The Table West Heading (total contract 4,204 m) "holed through" on August 21, 1983 and contractor was fully demobilized by October, 1983.

- The Table East Heading (total contract 4,844 m) "holed through" on August 21, 1983 and the contractor was fully demobilized by October, 1983.

- The Wolverine West Heading (total contract 2,533.5 m) "holed through" on May 28, 1983 and the contractor was fully demobilized by August, 1983.

- The Wolverine East Heading (total contract 3,378.8 m) "holed through" on May 28, 1983 and the contractor was fully demobilized by August, 1983.

With respect to the Km 80 and 86 Tunnels (in the Phase II study being referred to as Tunnels #4 and #5, Figure 1) 269 and 366 metres in length respectively, geological investigations and survey of the two sites were carried out in 1981. This included the excavation of several trenches, seismic infraction surveys, drilling and geologic mapping at the proposed tunnel sites. Tunnel support types were selected and possible distribution of support along the tunnel was determined. The design parameters were also prepared for cut slopes in rock and overburden at the portals. The drawings and specifications for the contract documents were prepared using the data obtained from the field program, and finalized by February, 1982. Construction commenced in September, 1982, with the excavation being concluded in December, 1982, and concluded in July, 1983.

Table 4 provides an overview of the planned (P) and actual (A) schedule covering design, tendering and construction of the Tunnels.

8. ELECTRIFICATION

8.1 Introduction

In 1968, a theoretical study was performed by B.C. Railway, covering existing railway operations between North Vancouver and Prince George. The study showed that the relative price of diesel fuel versus electrical energy should more than double to produce savings adequate to support the cost of capital, low as it was at that time.

Preliminary studies for access to the North East Coalfields touched on electrification in 1975/76. This was prompted by concerns over ventilation when long tunnels were considered in route alternatives.

Electrification was not pursued, partly because of estimates of the cost of increased rock excavation required for electrical clearances, erroneously based on wire heights often used on the old Eastern U.S.A. systems in open country.

There was no convincing modern electrification data of any kind that could be related to the type of heavy coal trains contemplated and the Railway assembled adequate operating plans based on diesel locomotives to support North East Coal development studies.

At about the same time, the Salt River Project Navajo power plant put a private 78 mile electric coal hauling line into operation in Arizona, energized at 50 kilo volts. Hitherto railways had not been worked at over 25 kilo volts.

Also, at that time, the U.S. Federal Railroad Administration (F.R.A.) commissioned a theoretical study of the potential for "dual mode" locomotives, i.e. a true hybrid locomotive with electrical capabilities equal to or better than its standard diesel power. The study contemplated piecemeal but progressive electrification along existing major routes starting wherever heavy traffic flows or heavy horsepower demands made electrification most financially attractive.

In Canada, a watching brief was sustained by certain members of the Railway Advisory Committee (R.A.C.), itself a link between the Railway Association of Canada and the Transportation Development Centre (T.D.C.) of Transport Canada. Development of electrified railways around the world was observed and it became apparent that the Swedish railway system operated in similar climate and topography to that of Western Canada. The very high pulling ability of the Swedish locomotive was also found to be very attractive for operating in mountainous terrain.

By membership in the Railway Association of Canada, B.C. Railway participated in the watching, but it was not until 1981 that the topic of electrification in Canada burst out from a half century of side talk. In 1981, several events occurred. The F.R.A. dual mode study was published and discussed by some R.A.C. members. Electrification was listed as a topic worthy of T.D.C. support. Peter Detmold, Chairman of R.A.C., publicized the use of electrification to overcome the speed limitations of diesels on heavy grades. B.C. Railway considered the use of dual mode for tunnel working due to the high capital costs involved in projected tunnel ventilation. B.C. Railway re-assessed the question of tunnel height for electrification.

B.C. Railway examined operating costs, capital savings, and possible assistance available from T.D.C. for electrification development and from Federal/Provincial Conservation and Renewable Energy programs.

Through T.D.C. support, utilizing a study that had been concluded in a proposed electrification program over a segment of a rail line in Eastern Canada, a feasibility review to electrify a portion of the Branch Line was carried out in early 1982. This study concluded that electrification would be feasible operating on a 50 KV system using Swedish technology.

On May 7, 1982, B.C.R. Directors were appraised of the favourable case for electrification, and permission was obtained to take a review panel of electrification experts over the entire Branch Line route.

With favourable indications from the review panel, funding agencies and locomotive suppliers, an immediate start was made to secure key components such as locomotives, power transformers, and manufacturing rights for Canada of the preferred Swedish hardware designs.

Formal ratification of all steps took several more months, but the process was smoothed by considerable enthusiasm for electrification at all levels and interim approval was always available for the immediate work in hand, recognizing the critical time schedule which was already a feature of the overall Branch Line project.

Economic analysis was very much site specific to the Branch Line. The key factor was eliminating the cost of ventilation fans and control gates needed for adequate air cooling of multiple diesel locomotives. However, as soon as the premise of electrifying the tunnels was tested, it became apparent that the necessary electrical

substation and power feed into the tunnels contained a fixed cost that could substantially support electrification of the entire Branch Line. Progressively, the concepts of tunnel electrification only, mixed diesel and electric locomotives, exclusive electric locomotives, and maximum extension of power supply to main line connection at Anzac, were found to improve the overall rate of return for added increments of investment. Improved returns came from the well known energy cost advantage of electrification on heavy fuel usage and the reduced maintenance costs of electric locomotives. The availability of funding through the Federal/Provincial Conservation and Renewable Energy programs was also a significant factor.

8.2 Power Supply

1) Substation - Primary power for the Railway is taken at 230 kilo volts (kV) from a B.C. Hydro and Power Authority substation located a few kilometres outside Tumbler Ridge Townsite and built as the feed point for all industrial and municipal activity in the area. Four radiating feeders serve the two active minesites at 230 kV and the townsite at 25 kV. The incoming Hydro feed is directly connected at 230 kV to Gordon Shrum hydroelectric station on the Peace River.

   The Railway substation will be built close alongside the Hydro substation. Thence the Railway supplies 50kV single phase power by a dual 2 km pole line feed to the rail line.

   Inside the railway substation, primary 230 kV power is connected via an ASEA 230 kV 3 phase minimum oil breaker to a 3 phase bus. Need for this breaker was found to be as much a legal convenience as technical need; a matter bearing on the contractual policies of B.C. Hydro toward close coupled protection and control schemes.

   For the foreseeable future, the railway will only load one phase at 230 kV. Parallel CGE 230/50 kV transformers, each 24/33 MVA discharge through ASEA 50 kV, SF6 breakers to a 50 kV single phase bus. Transformers and bus can be segregated by motor disconnects. Either transformer can handle present traffic load. The 50 KV bus feeds two outgoing circuits and a 5 MVA harmonic filter bank via SF6 breakers and disconnects.

   Protective relaying and supervisory control is remote, in the Railway's Dispatching and Power office in North Vancouver. The office equipment comprises a passive mimic diagram with indicating lights and an active colour CRT display and keyboard with event logging printers. Control is also possible on site in the substation relay building. Remote control is achieved via the Railway's established microwave communication and data network. North Vancouver power office also has control of certain electrical disconnect switches used for sectioning the Branch Line for repairs or troubleshooting.

2) Catenary - Supply of 50 kV power to trains along the rail line is achieved by an overhead contact system commonly referred to as "the catenary" (Figure 14). Actual hardware comprising the catenary is derived from Swedish designs refined for Canadian use at 50 kV.

Lightweight steel masts, commonly referred to as "poles" are fabricated from two channels in a tapered form. The poles are bolted to a variety of foundations developed by B.C. Railway to suit site specific subgrade conditions on the Branch Line.

Foundations include driven steel piling, preformed concrete cylinders and bolts anchored in rock. With a sensitive subgrade and no rails in place, every effort was made to minimize excavation and site poured concrete.

Overhead contact wire is supported from poles by cantilevers consisting of insulators in upper and lower members; the members being steel rod or tubes all connected via slip fitted malleable iron fittings locked in place by set screws. All iron and steel is galvanized. Support clamps for the contact wire are made of cupro nickel alloy.

The contact wire of hard drawn copper with 107 mm2 ASTM trolley wire section is supported between poles by a 70 mm2 hard drawn stranded copper messenger and a series of droppers. Droppers are assembled from copper wire and cupro nickel clips. Maximum pole span is 70 m, a distance that can only be achieved on tangent track. The contact wire is staggered along track centreline by 400 mm each side at alternating poles. Chording to this offset on curves determines the permissible pole span around curves. Poles may be set inside or outside curves, but outside is preferred for ease of wire stringing and cantilever maintenance. In general, outside location also simplifies foundation details.

A ground current equalizing wire of 4/0 ACSR section is strung along all poles. This wire is bonded at intervals to rails and a pole base. Bonding frequency is determined from site specific measurements of ground resistivity in relation to safe levels of rail potential in proximity to locomotives and electrical faults. Suitable grounding is also achieved at the substation transformers.

A supplementary 50 kV feed wire of 4/0 ACSR is also strung along the poles on standoff insulators for much of the route. This wire is jumpered to the contact and messenger wires at intervals and provides increased conductivity without change in materials or weight of the catenary system.

3) Series Capacitor - A significant part of the voltage "loss" in any alternating current transmission line is reactive loss, i.e. change of volt/current phase relationship. This change can be annulled quite effectively by running the transmitted power through a "capacitor" at discrete intervals along the line. Capacitors can take several forms; electromechanical (rotary) and static.

One such capacitor of the simple static type is installed on the Branch Line catenary at Km 67 between the tunnels. This is a convenient operating location as locomotives must be dead between the phase change. The capacitor effectively reduces volt drop at all points on line remote from the substation, i.e. west of Km 67. Use of such a capacitor is part of the economic trade off in system design.

8.3 Locomotives

1) Choice of Manufacturer - Since the turn of the century, electric locomotives have been manufactured in the U.S.A. For railways such as the New Haven, Milwaukee, and Pennsylvania. By the late '30's, companies such as Baldwin, Westinghouse, and General Electric had all produced electric locomotives of heroic proportions, articulated units outweighing and out-axling the largest steam locomotives. These locomotives employed a variety of motor designs and other electromechanical apparatus totally foreign to most power engineering students today. By today's standards, such locomotives would be expensive to build and quite labour intensive to maintain.

By the late '50's, in the heyday of diesel locomotives application, only General Electric offered electric locomotives using the physical style of the diesels. By the late '60's, G.E. had adopted semiconductor rectifiers with motors similar to diesels and by the early '70's, thyristors appeared to eliminate most transformer switching.

In the early '70's, the Electromotive Division of General Motors (E.M.D.) who had achieved a world sumpremacy in diesel locomotive production, decided to establish an electric locomotive product. Like the Canadians, they perceived Sweden as a fitting technical and business partner and entered into licensing arrangements with ASEA of Sweden. Two speculative prototype locomotives were built which were aimed at North American freight haulage.

Due to disappointing performance of G.E. electric locomotives on AMTRAK's high speed passenger trains in the North East Corridor, the former Pennsylvania R.R., AMTRAK decided to hold a competition open to world builders. French and Swedish locomotives were actually put into service on the Corridor for evaluation. The Swedish locomotive was considered outstanding, and E.M.D. as ASEA's licensee subsequently received orders for locomotives, which were built in Chicago.

When B.C. Railway solicited bids for electric locomotives in 1982, both G.E. (U.S.A.) and E.M.D. (G.M. of Canada) were able to offer current designs. G.M. was selected as both commercial winner and for having a superior sustained tractive effort, achieved through their method of motor connection, and a primary consideration for heavy pulling out of Tumbler Ridge up to the summit tunnels.

B.C. Railway locomotives are 178 tonnes gross weight on six individually motored axles. Motors are four pole direct current commutator type with interpoles and separately excited uncompensated fields, G.M. model E88. Armature coils are transposed within the slots. The motor frame is suspended between truck (bogie) frame and axle using roller type axle suspension bearings.

8.4 Design and Construction

Design of the electrification system commenced in June, 1982 involving B.C. Railway and a consortium of consultants representing C.N. Rail, C.P. Rail, and Swedish State Railways. Currently the designs are completed including the preparation of drawings, specifications, and contract documents covering the complete overhead contact system, hardware, substation installation and equipment, protection and supervisory control, series capacitor station, poles, and pole installation. Contract documents for the installation of the overhead contact system was prepared and was tendered by May, 1983, with contracts awarded in July, 1983.

Installation of the poles commenced in May, 1983 in the Wolverine River Valley, with the installation in the Table River Valley commenced in June, 1983. Installation of the overhead contact system commenced in July, 1983 and was concluded by December 15, 1983 between Km 24 and Km 130, with the remaining portion scheduled for 1984. Energization commenced in November, 1983 and was fully energized by February, 1984.

9. TRACK

The standards for track material adopted in the Phase II study have been maintained, with the exception of weight of rail being changed from 136 lb. to 115 lb. The initial plan was to place Continuous Welded Rail as soon as it became possible to lay track, which was mid May, 1983. In consideration of the production that could be achieved in placing CWR on newly built grade, its associated cost and the severe time constraints, an alternate plan was considered. The alternate plan was to first lay jointed rail (78 foot lengths) followed by relaying with CWR at a later date. Based on production rates projected to be achieved on a daily basis for both alternatives, taking into consideration work force experience, the resultant cost analysis produced heavily favoured laying jointed rail in 1983 and relaying with CWR a year later. This alternate proposal became feasible and attractive due to the following four reasons:

1) Productivity per day would be higher, therefore the cost per kilometre would be reduced.

2) B.C. Railway have on hand an adequate quantity of partly worn 100 lb. and 115 lb. rail and new 100 lb. rail.

3) The schedules were extremely severe, and should the tunnels be only delayed slightly, say one month, using CWR the completion of December 1, 1983 would not be met. By placing jointed rail, alternate plans would be possible such as transporting all track material by truck from Chetwynd to the Wolverine River Valley, and thus permit trackwork to proceed concurrently on either side of the tunnels.

4) Quality of trackwork would be ensured.

Rail laying and placement of ballast commenced in June, 1983, and was concluded on October 21, 1983. Relaying with CWR is scheduled to commence in June, 1984, including the conclusion of placement of ballast.

Reference should be made to Table 3 for a brief overview of the track construction schedule.

10. CONCLUDING REMARKS

The British Columbia Railway takes pride in being one of the participants in the development of the North East Coalfields. The time constraints were severe for all participants, which presented a unique challenge. With respect to the Tumbler Ridge Branch Line, the current projections indicate that the costs at completion will be within budget. The first coal train, using diesel electric locomotives departed the loadout loop (Teck Corp. at Km 115) on November 1, 1983.

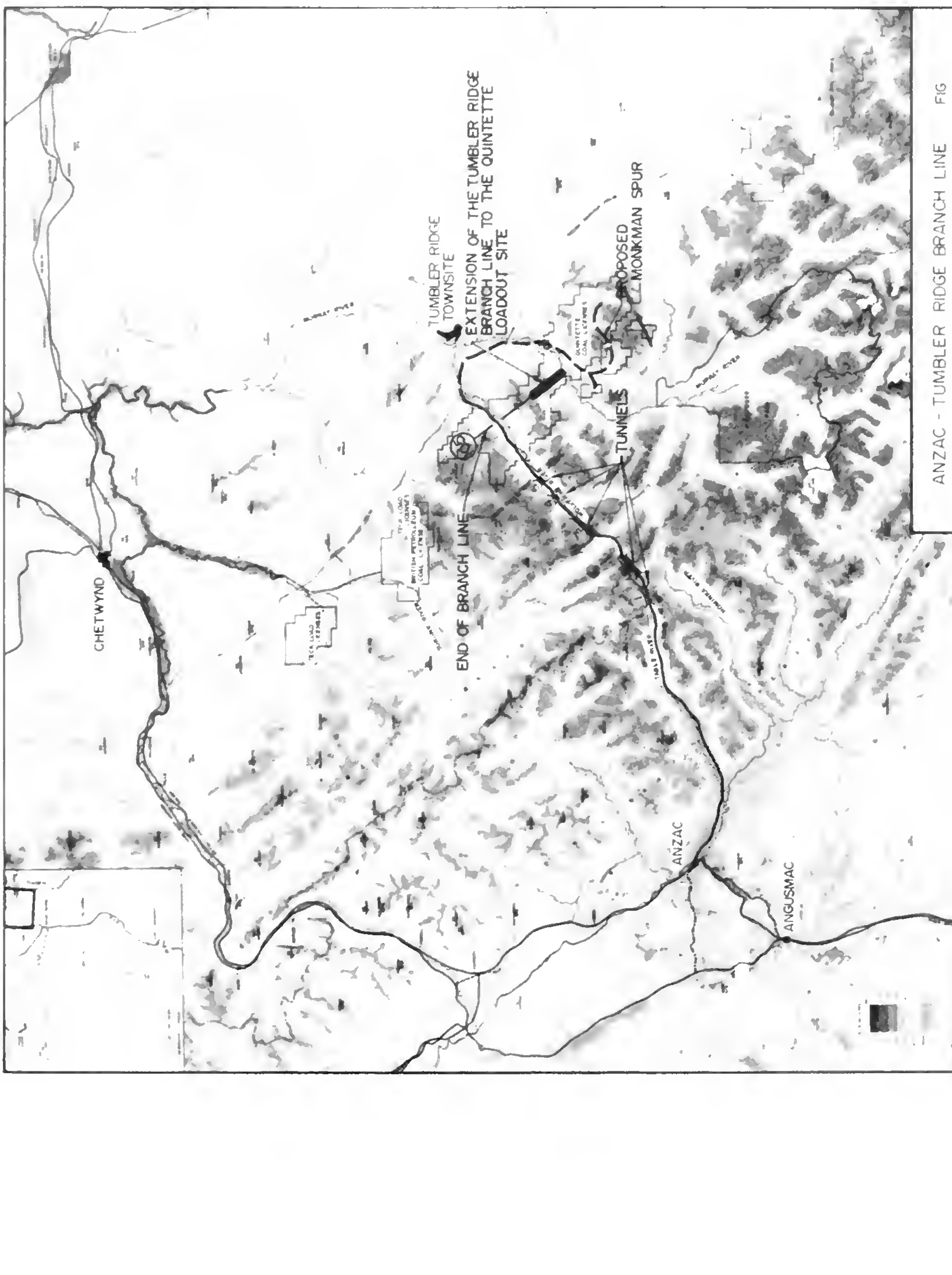


FIG. 1. Anzac—Tumbler Ridge Branch Line

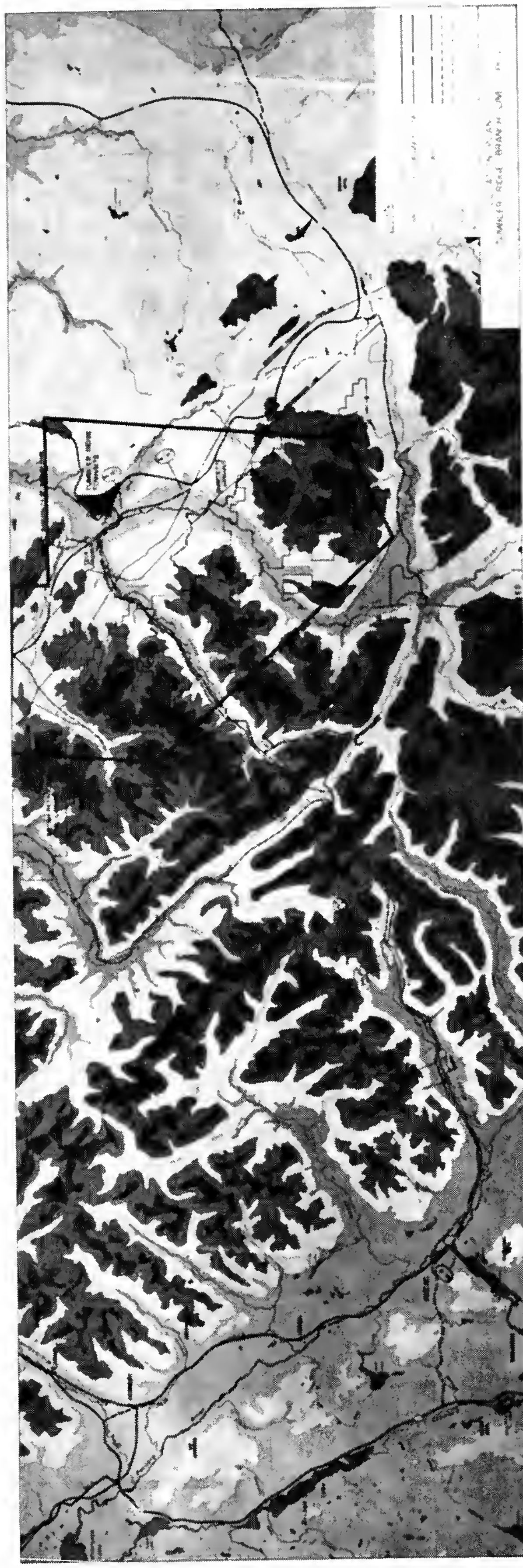
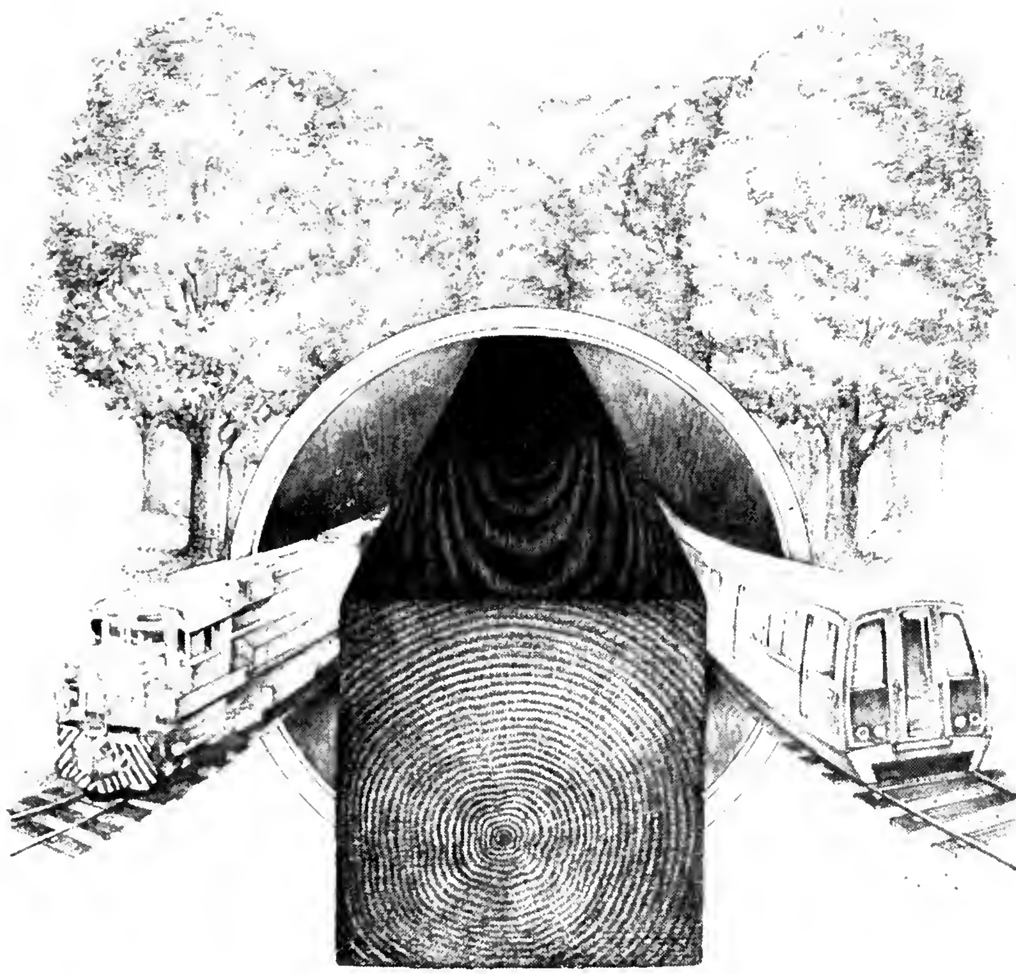


FIG. 2. Tumbler Ridge Branch Line—Location Plan



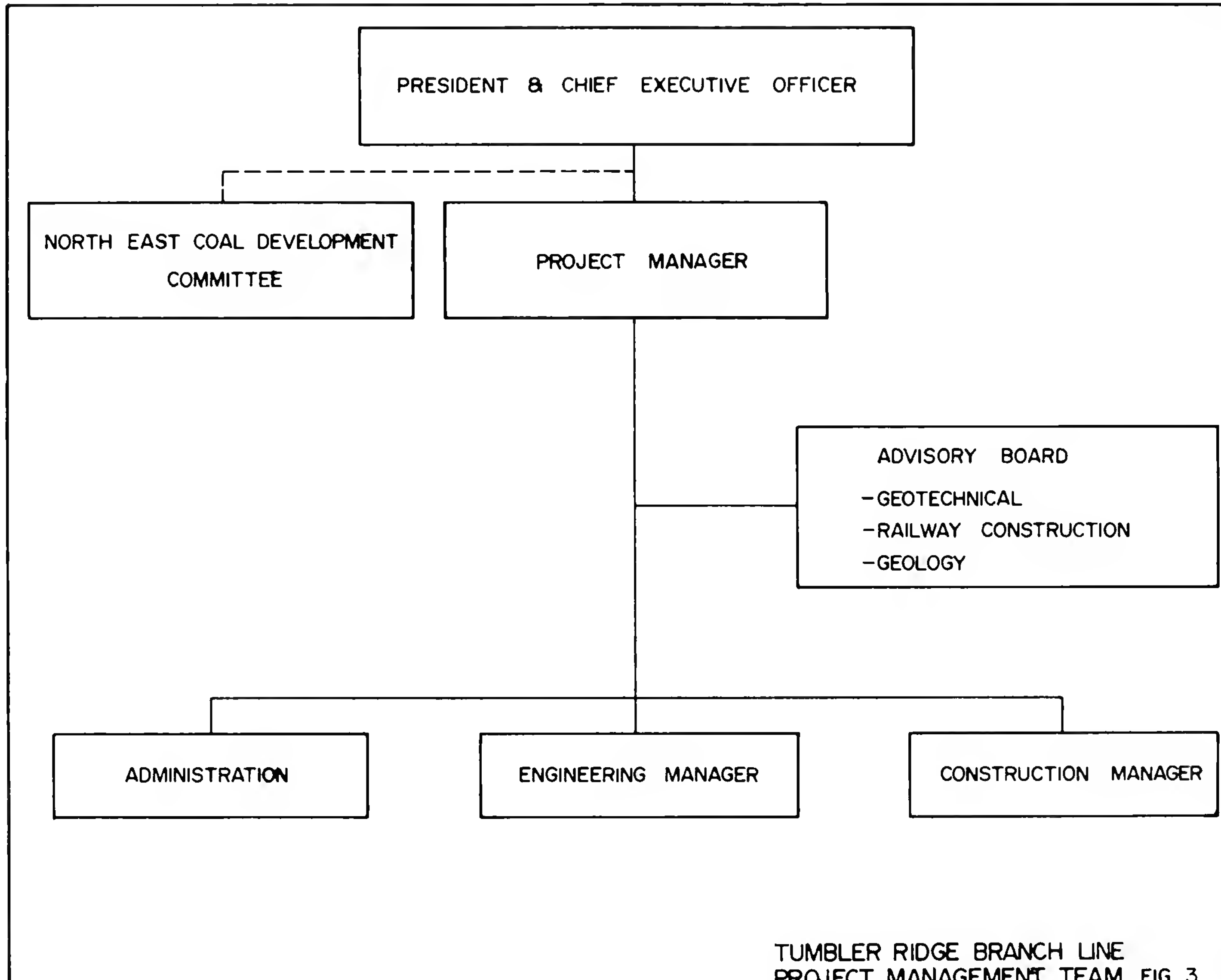


FIG. 3. Tumbler Ridge Branch Line—Project Management Team

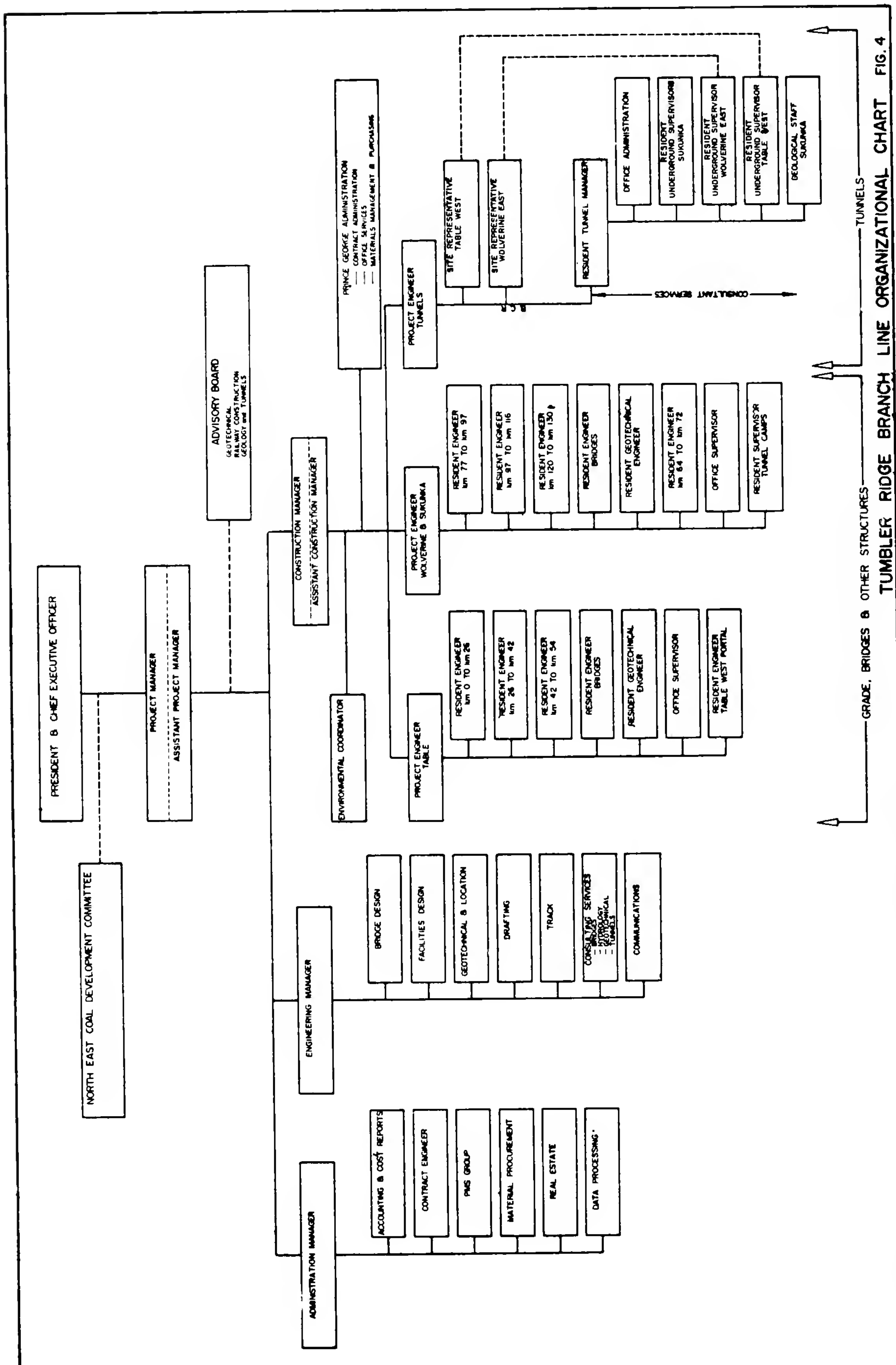


FIG. 4. Tumbler Ridge Branch Line—Organizational Chart

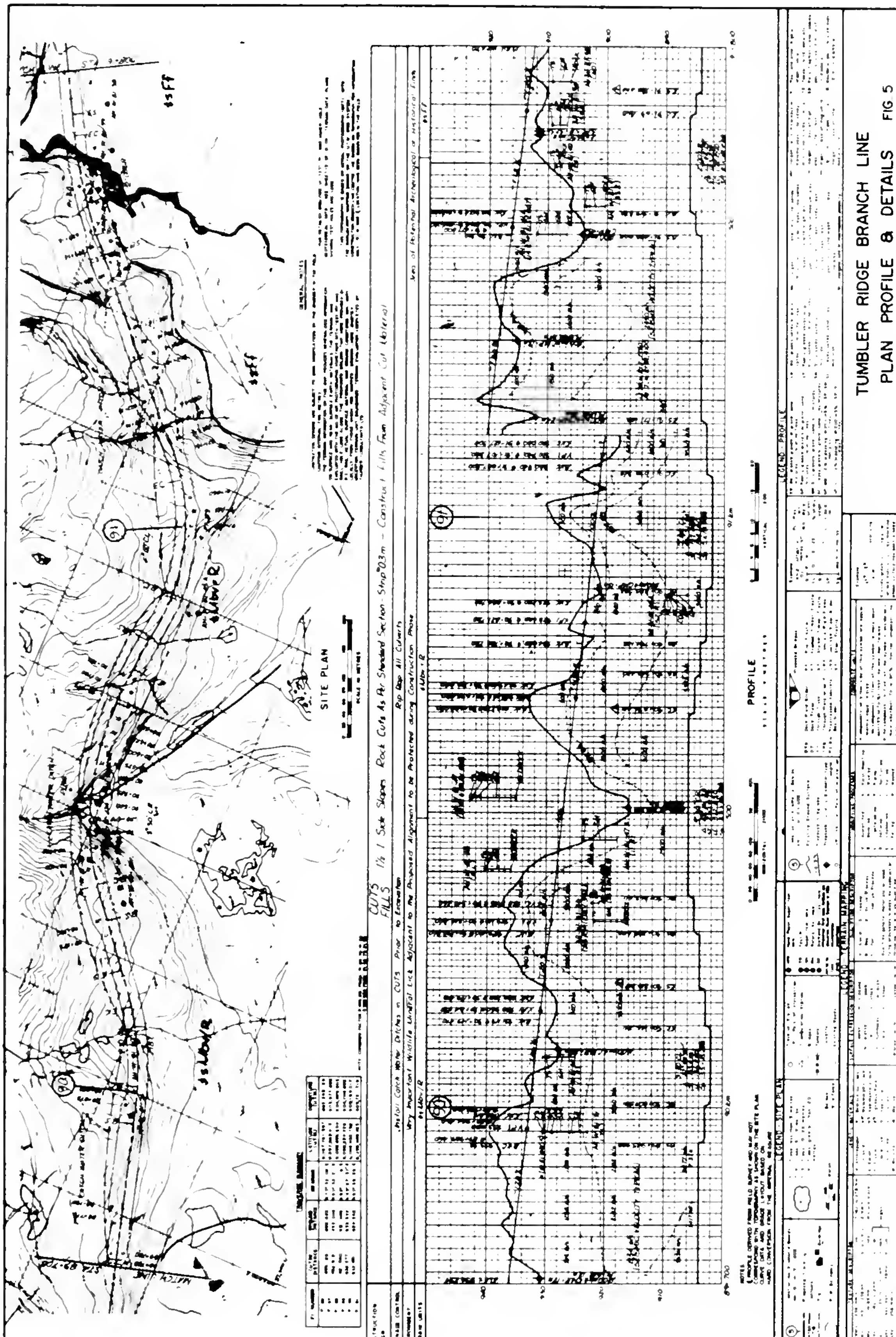


FIG. 5. Tumbler Ridge Branch Line—Plan Profile & Details

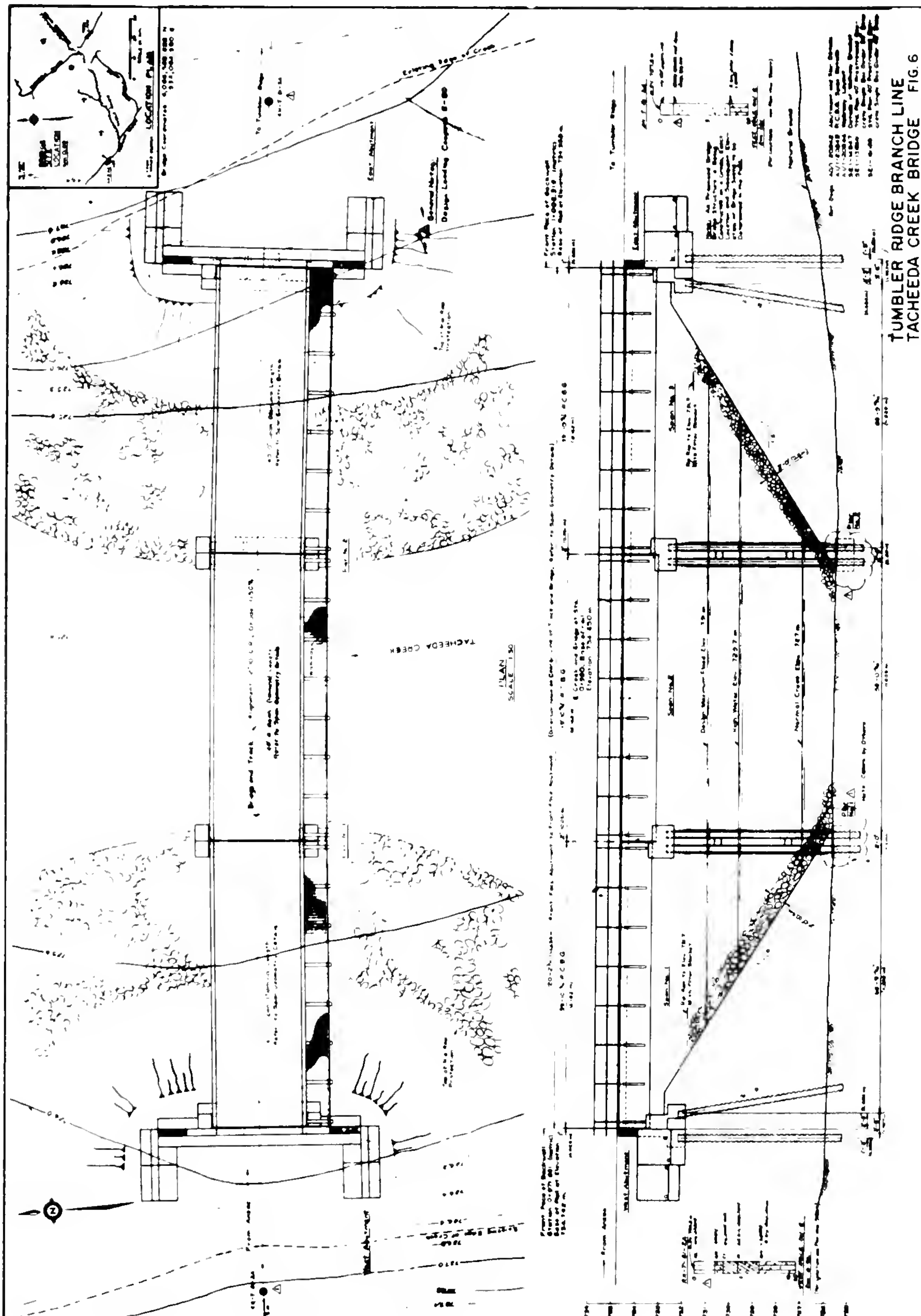


FIG. 6. Tumbler Ridge Branch Line—Tacheeda Creek Bridge

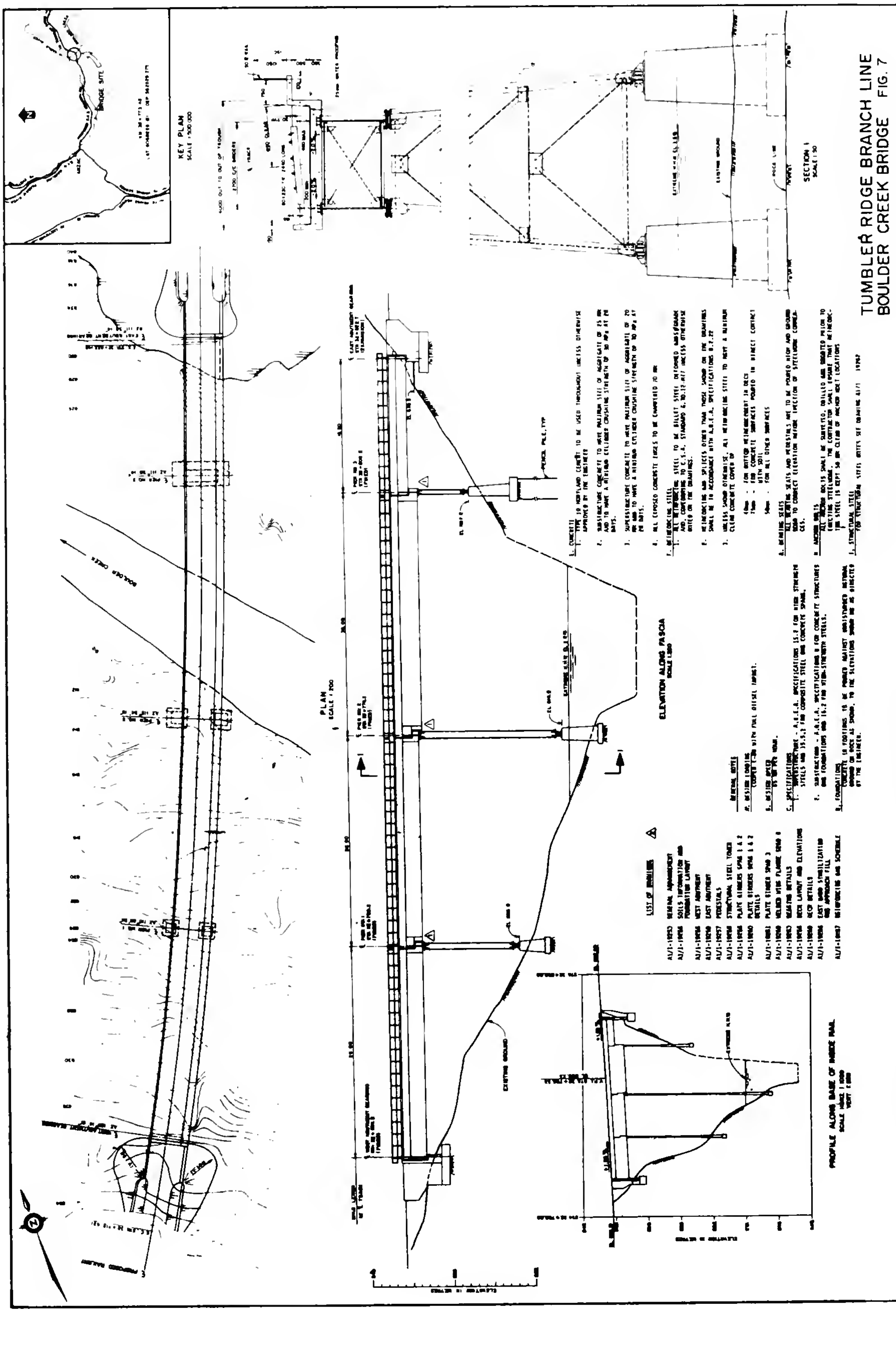
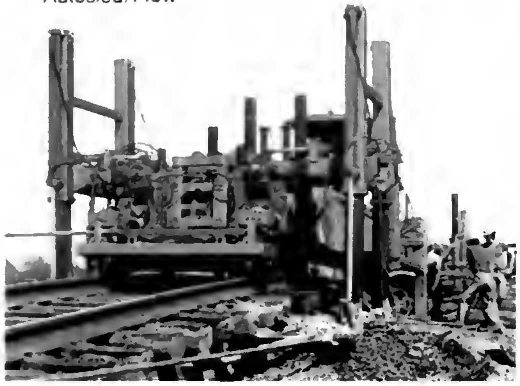


FIG. 7. Tumbler Ridge Branch Line—Boulder Creek Bridge

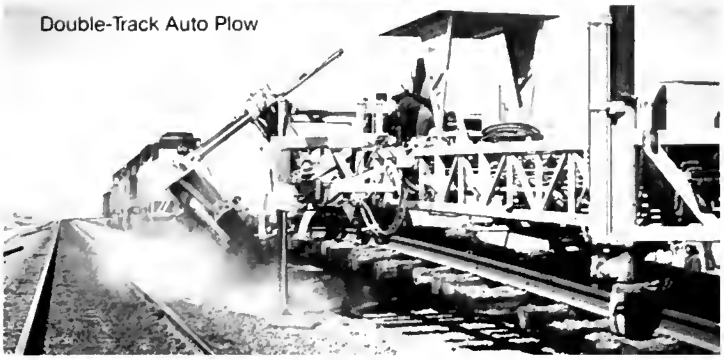
Shoulder Ballast Cleaner

Winch Cart

Autosled/Plow



Double-Track Auto Plow



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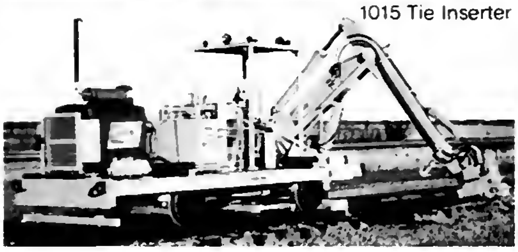

SP-88 Rail Grinder

LORAM'S ALL-PRO TRACK REHABILITATION TEAM

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LORAM RAIL GRINDERS (24-, 36-, 72- and 88-stone models) grind down to the rail corrugation valleys instead of *into* them, as other grinding methods do. Loram grinders restore rail without wasting rail metal.

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LORAM'S AUTOSLED/PLOW, with plowing and sledding components built right in, can be set up fast—actually in about 11 minutes.

LORAM'S TIE INSERTER inserts five or more ties a minute and *can be easily adapted to handle concrete ties.* Design simplicity and very accessible parts make the 1015 easy to maintain and repair.

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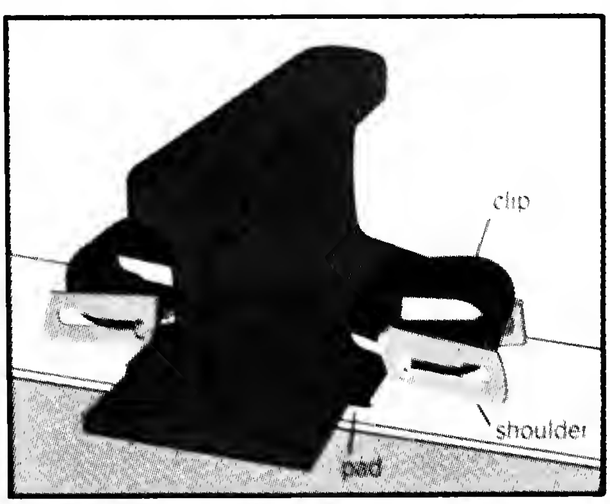


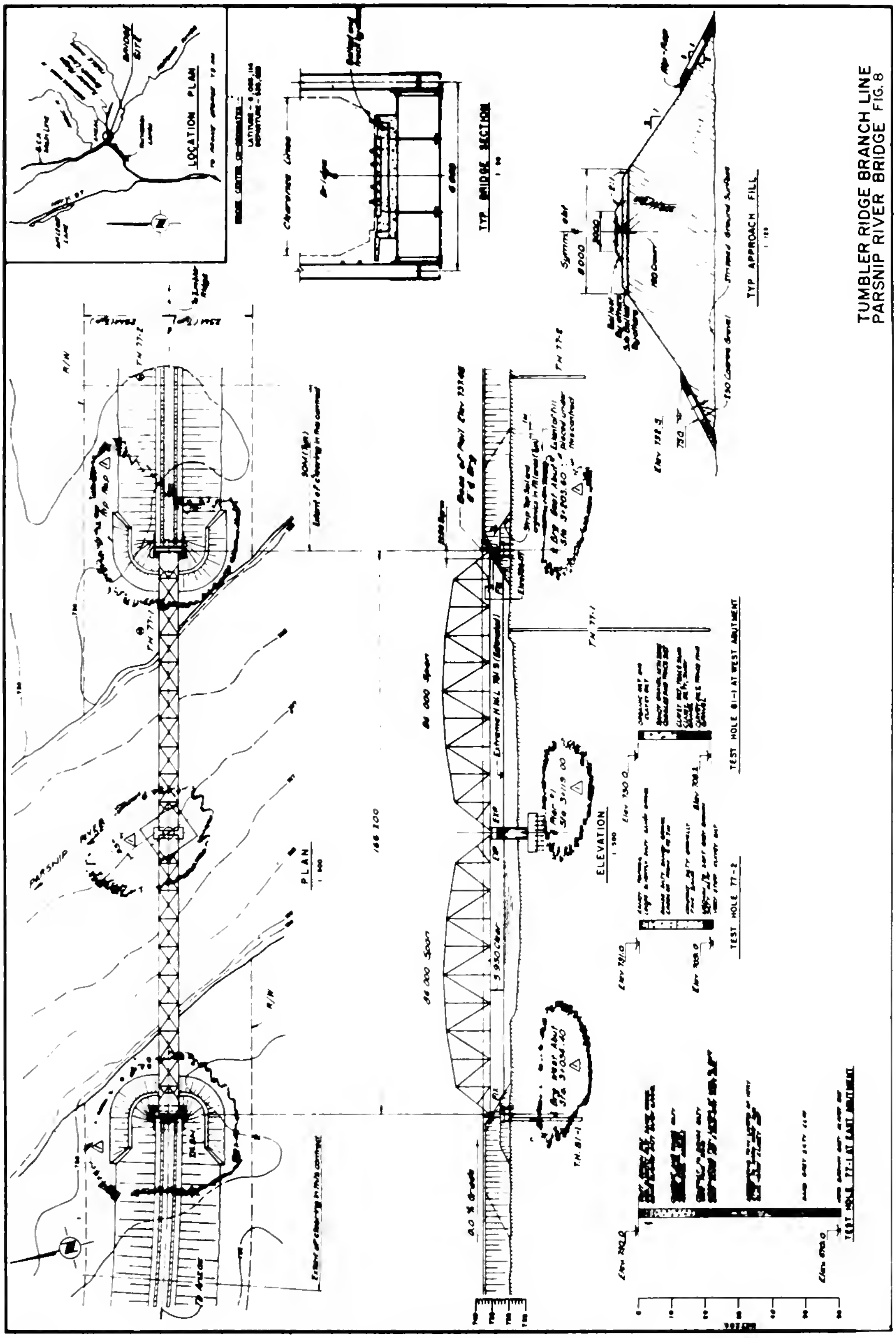


FIG. 8. Tumbler Ridge Branch Line — Parsnip River Bridge

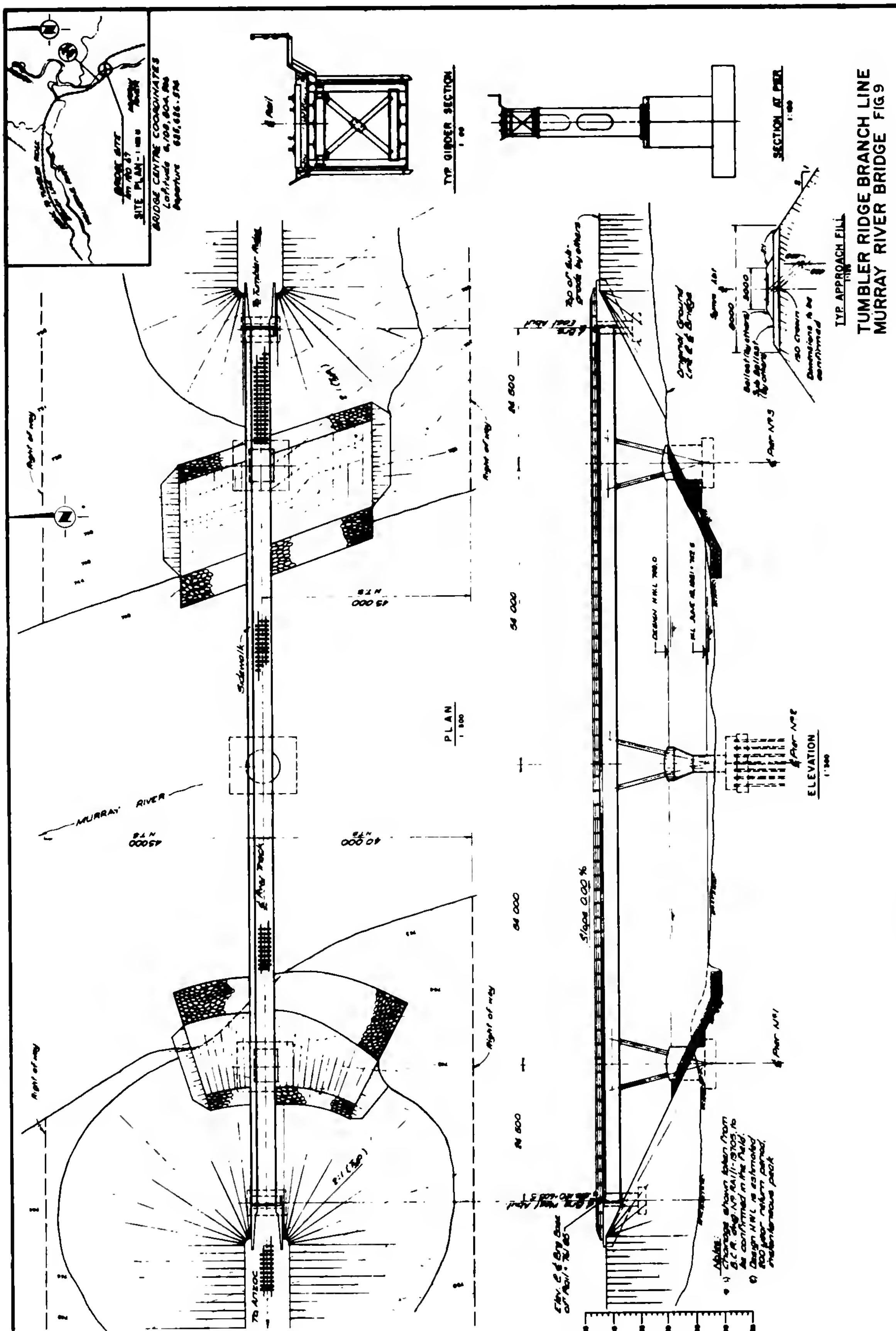


FIG. 9. Tumbler Ridge Branch Line—Murray River Bridge

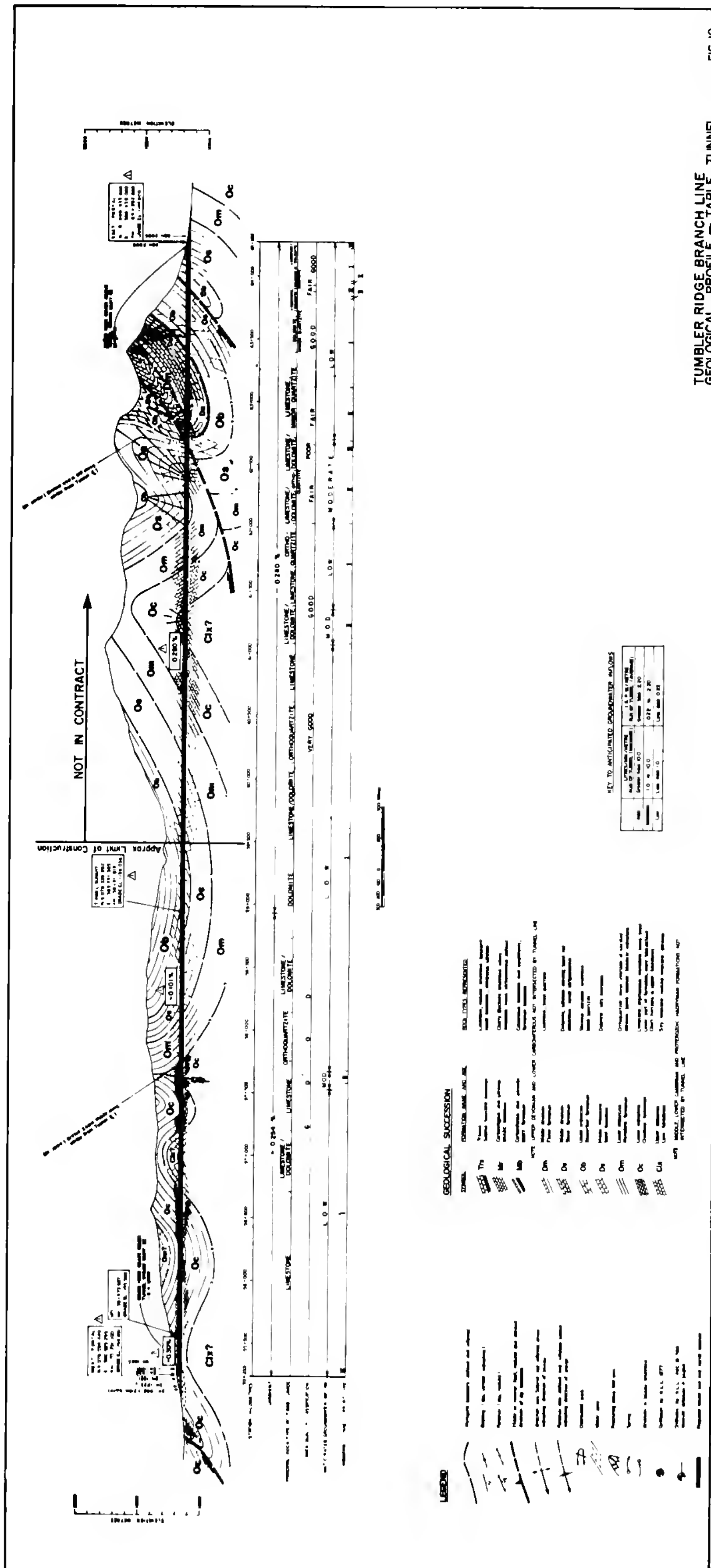


FIG. 10. Tumbler Ridge Branch Line-Geological Profile—Table Tunnel

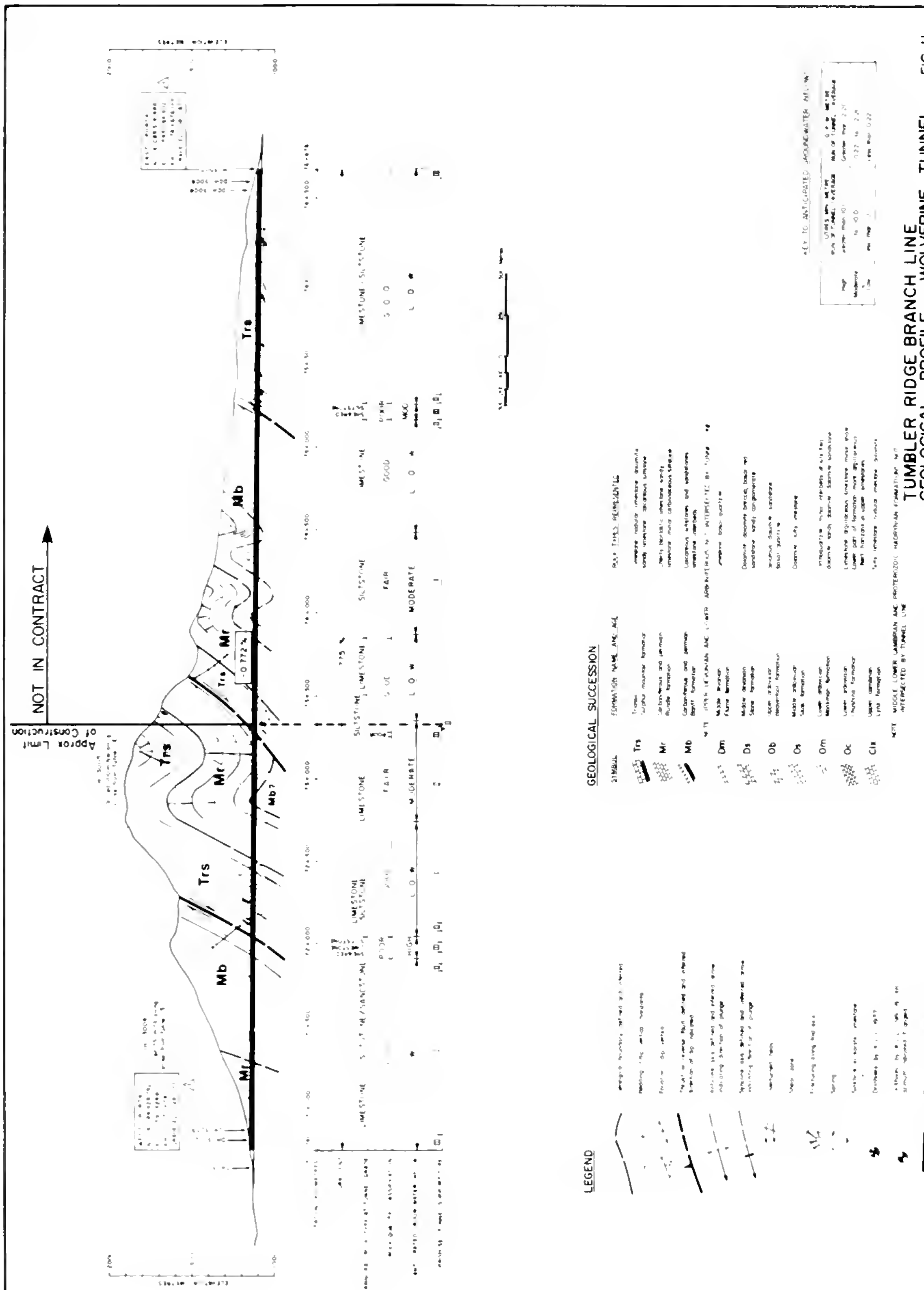


FIG. 11. Tumbler Ridge Branch Line—Geological Profile—Wolverine Tunnel

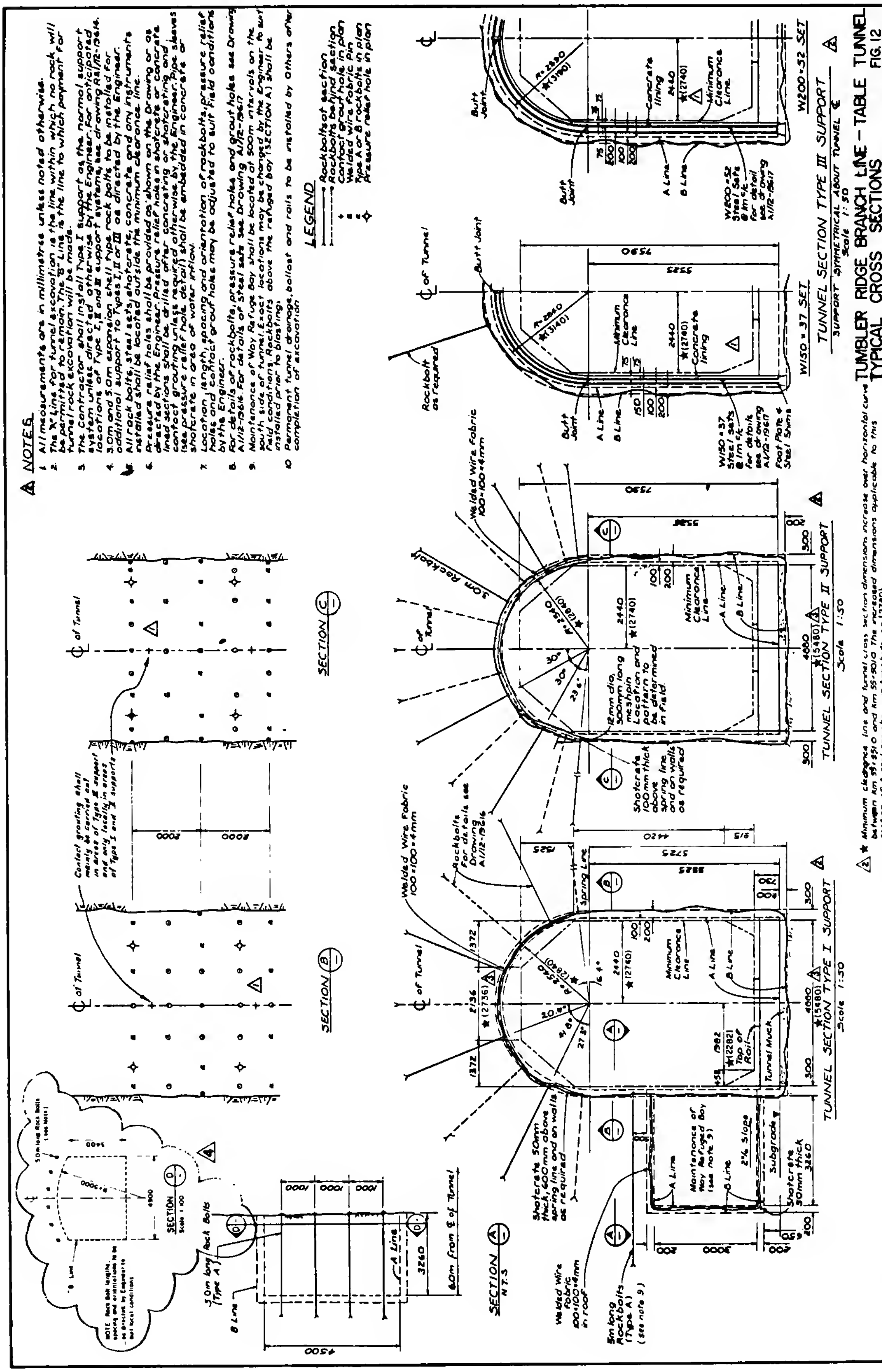


FIG. 12. Tumbler Ridge Branch Line—Table Tunnel Typical Cross Sections

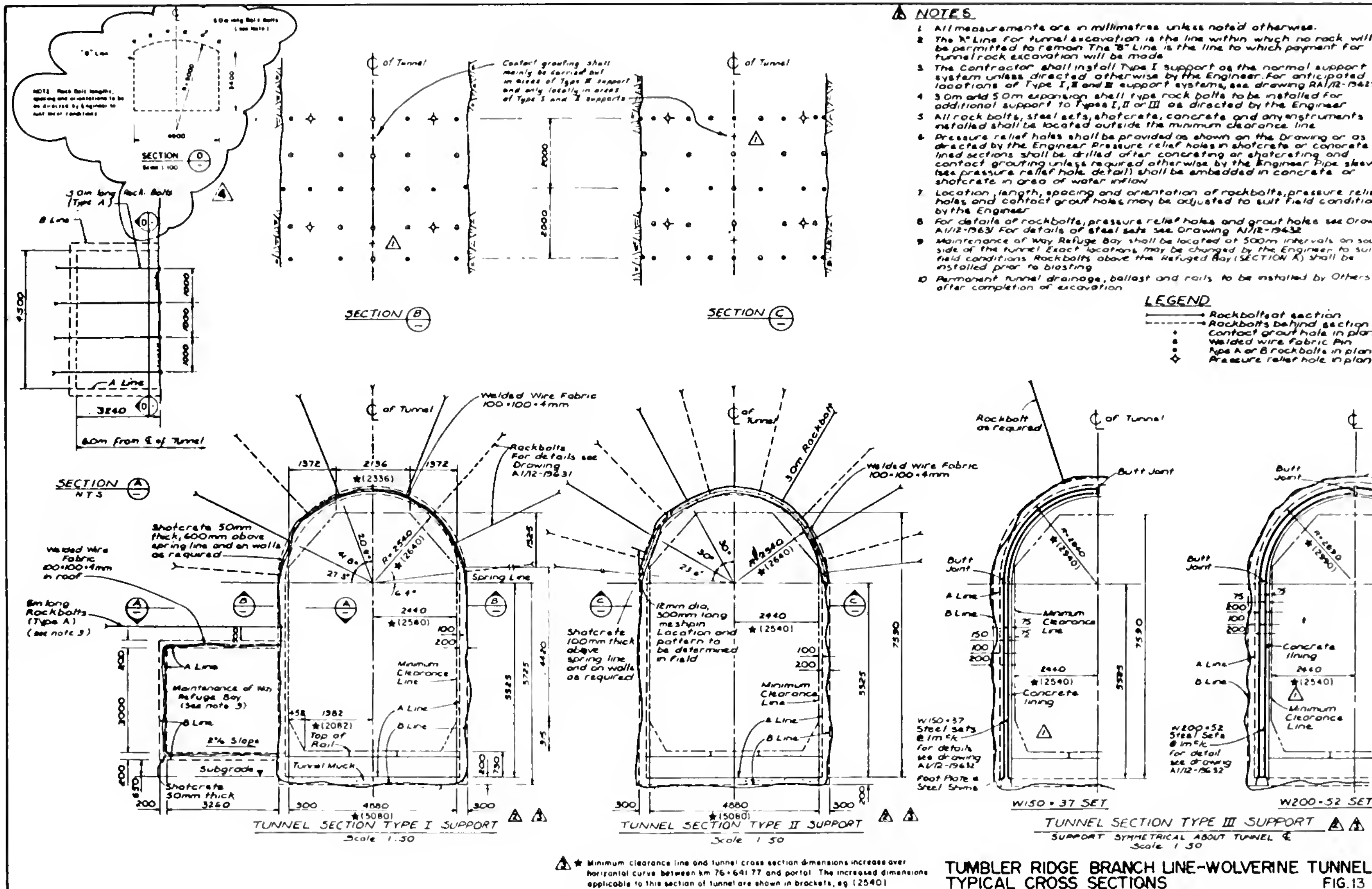
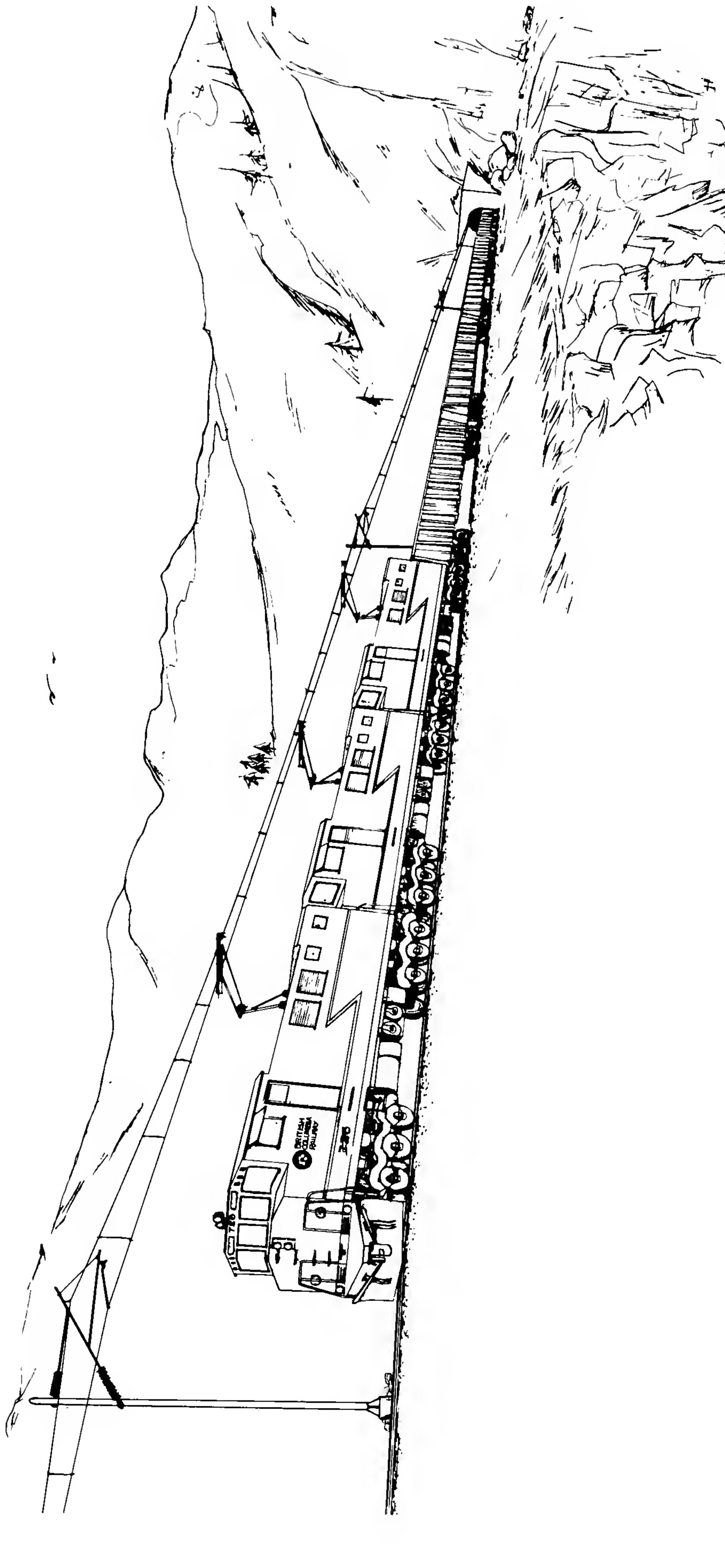
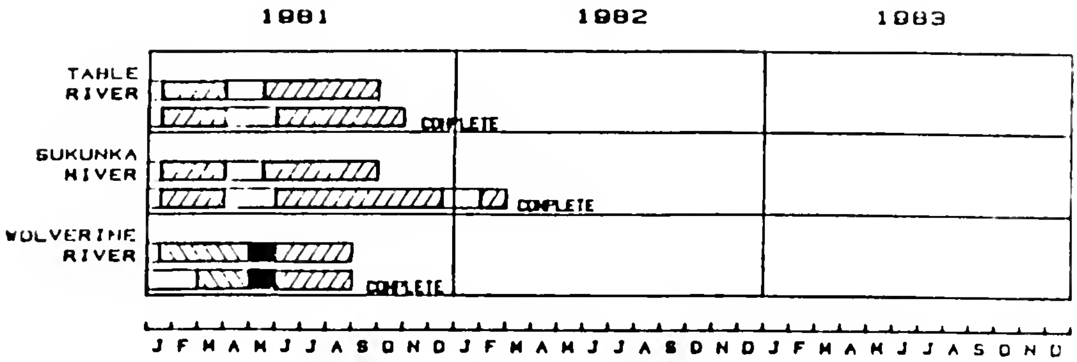


FIG. 13. Tumbler Ridge Branch Line—Wolverine Tunnel Typical Cross Sections

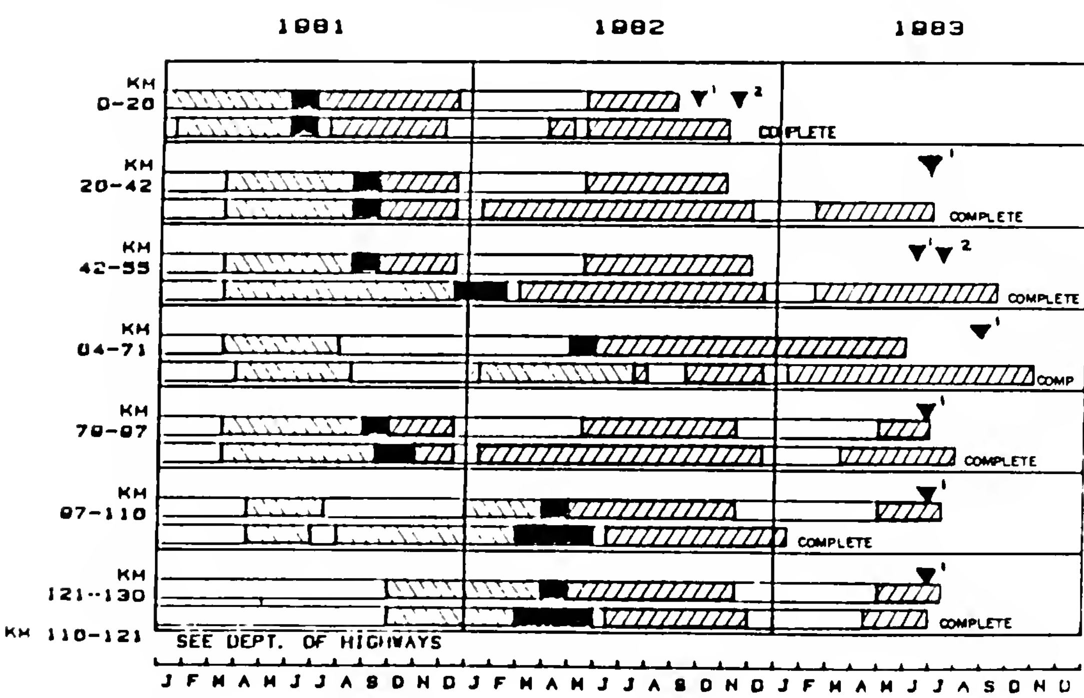


TUMBLER RIDGE BRANCH LINE
ELECTRIFICATION - SCHEMATIC FIG. 14

Access Roads



Grading



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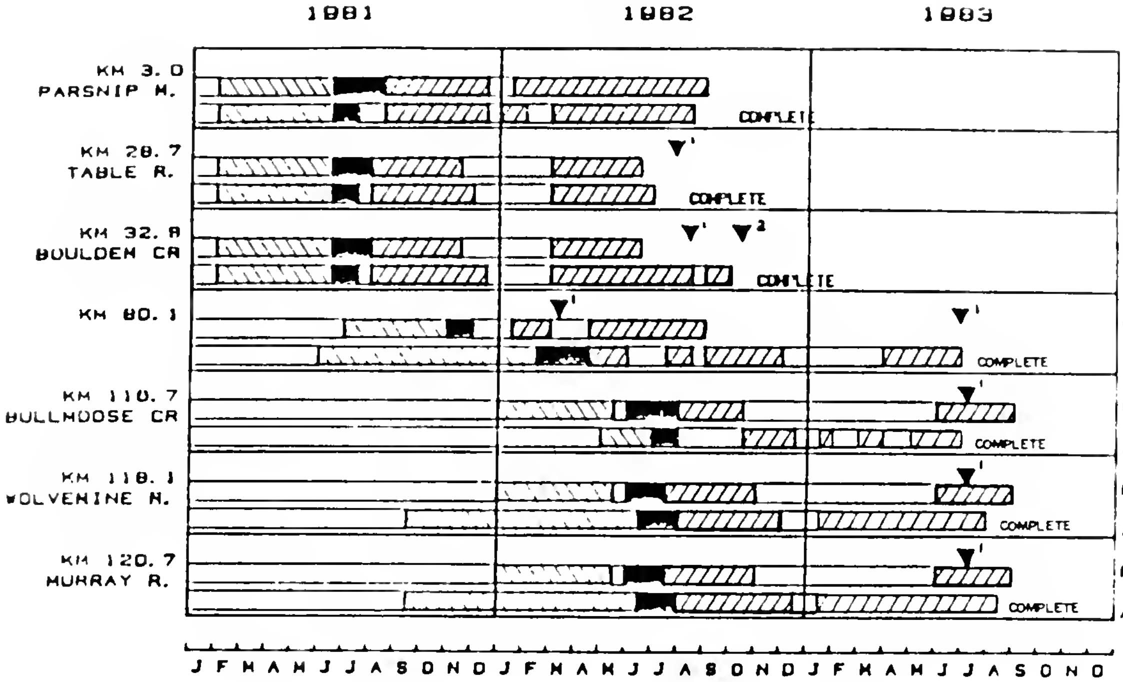
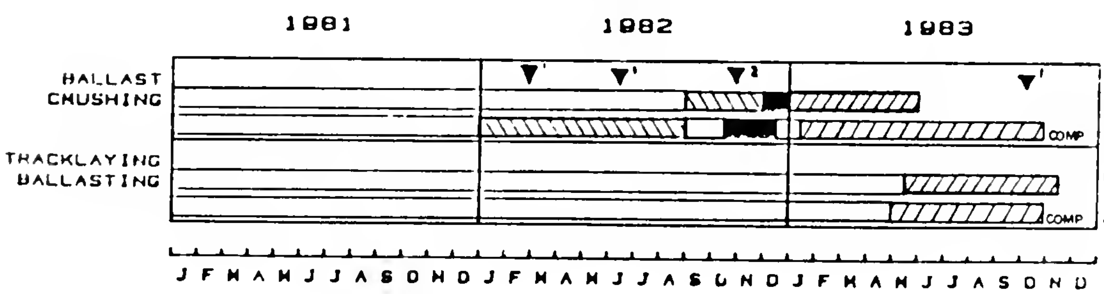
DESIGN & CONSTRUCTION SCHEDULE TABLE 2

Bridges



Track



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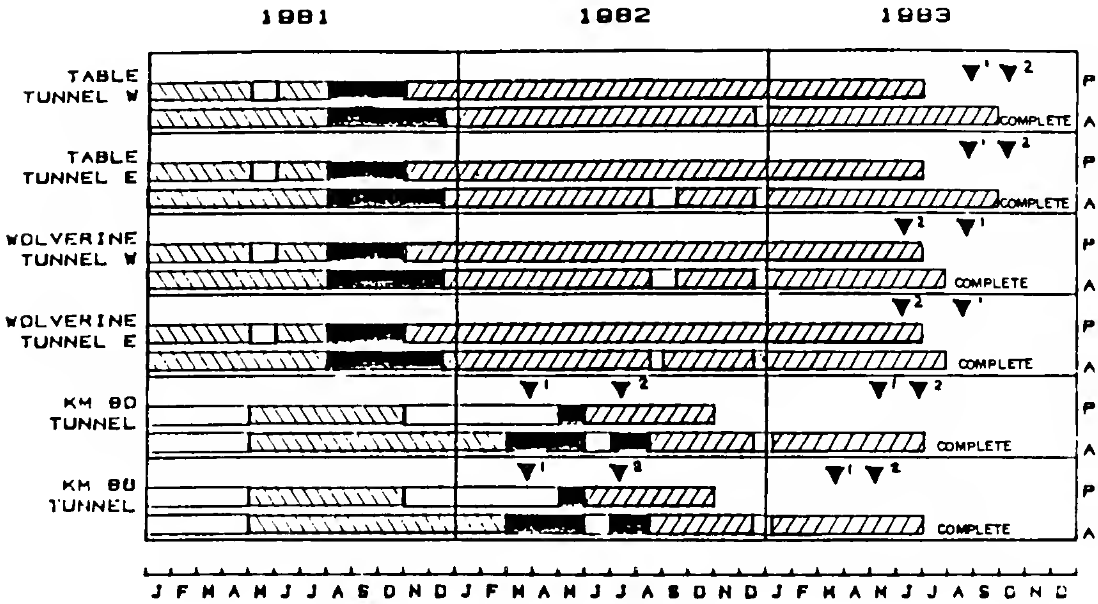
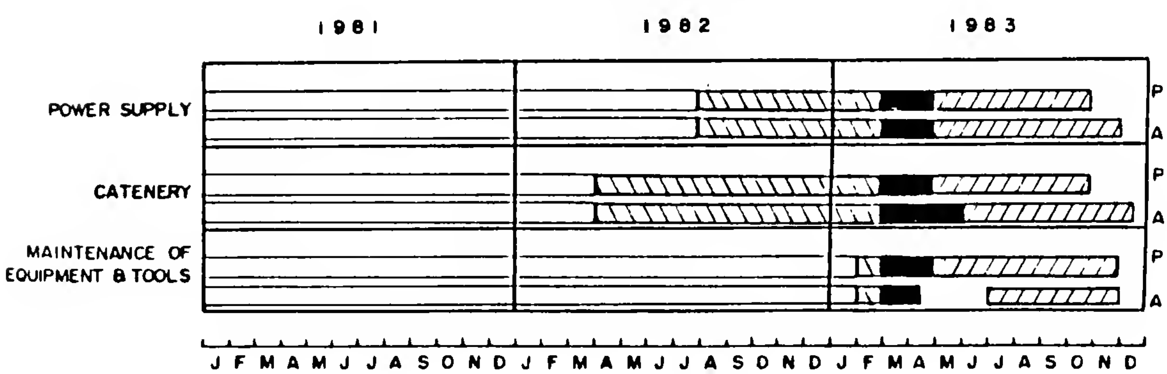
DESIGN & CONSTRUCTION SCHEDULE TABLE 3

Tunnels



Electrification



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DESIGN & CONSTRUCTION SCHEDULE TABLE 4

PHOTO. 1 Topography in the vicinity of the Sukunka Valley looking towards the Wolverine Valley.

PHOTO. 2 View of completed track and overhead contact systems in the Table River Valley.

PHOTO. 3 View of grade under construction looking easterly towards the Table Tunnel.

PHOTO. 4 View showing bundles of ties distributed along completed grade in the Table River Valley.

PHOTO. 5 Grade under construction in the Sukunka River Valley.

PHOTO. 6 Aerial view of grade and track in the Wolverine River Valley.

PHOTO. 7 View showing completed grade, track and overhead contact systems in vicinity of Km 97 (Wolverine River Valley).

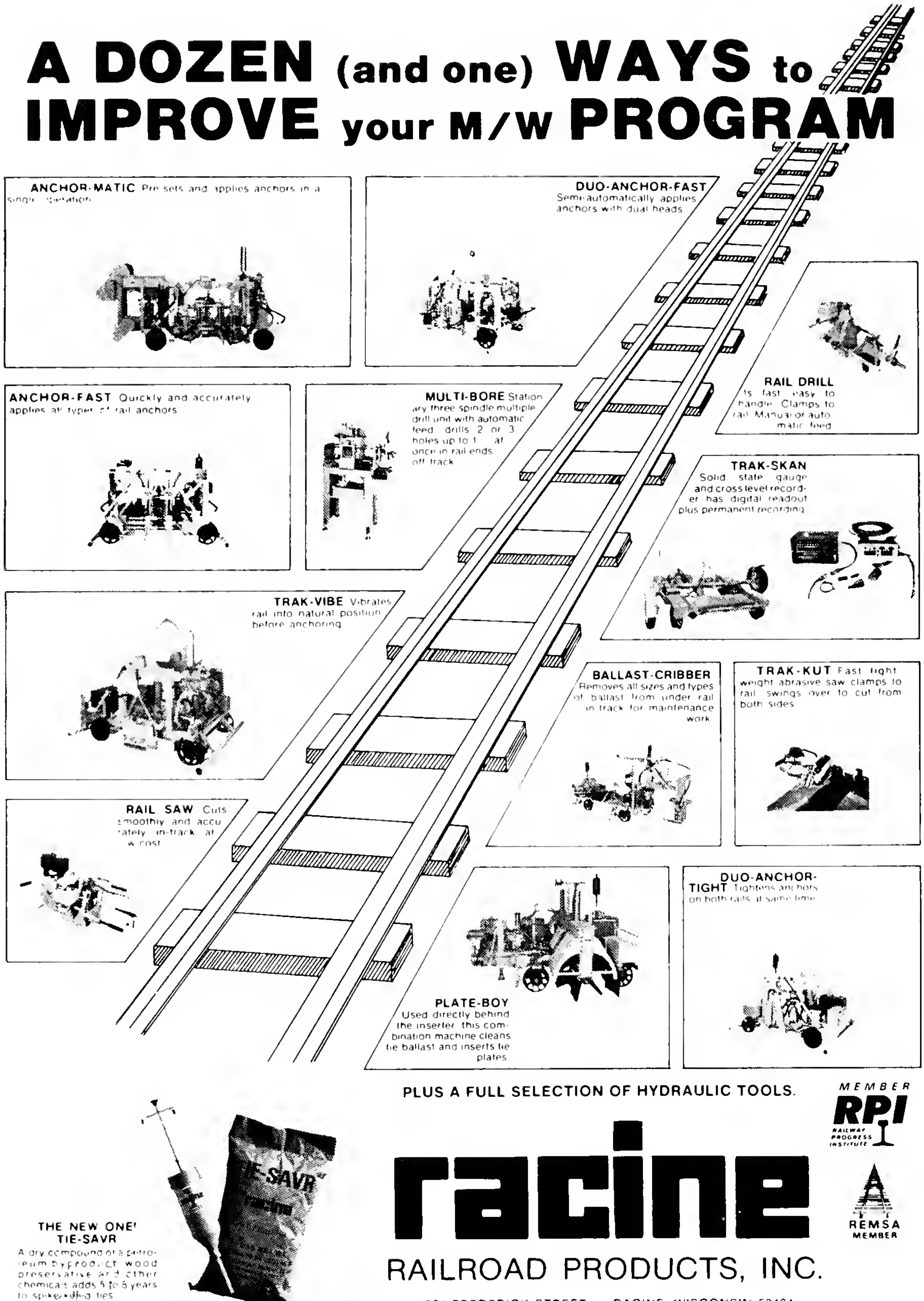
PHOTO. 8 View showing placement of wire mesh in the construction of the Table Tunnel.

PHOTO. 9 The Parsnip River Bridge being launched (July, 1982).

PHOTO. 10 Aerial view of completed Lost Creek Bridge at Km 80.

PHOTO. 11 Aerial view of completed Bullmoose River Bridge in vicinity of Km 116.

PHOTO. 12 View showing Murray River Bridge being launched into place.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COMMITTEE 5—TRACK

"Performance Characteristics for Wood Tie Fasteners"

Allan M. Zarembski*

I INTRODUCTION

As railroad freight axle loads and train weights have increased, there has emerged growing concern as to the adequacy of conventional cut spike fasteners for wood tie track. This concern, coupled with the advent of a new generation of wood tie fasteners, has led to a requirement for the development of performance characteristics for wood tie fasteners under present heavy axle load railroad conditions.

Over the years, the performance of cross-tie fastener systems has been the subject of numerous analyses, investigations and testing. From these investigations, have emerged the range of performance characteristics that are required for fasteners under heavy traffic and axle load conditions. These characteristics differ from the European and other lighter axle load applications in that the load environment under which the fasteners (as well as the ties) must perform effectively is significantly more severe in all major areas: vertical loading, lateral loading, and longitudinal loading. This is particularly true in many of the North American freight railroads' non-conventional fastener applications to date. These have concentrated on the severe curvature territories that had been traditional problem areas for wood ties and cut spike fasteners. These areas in turn are the ones in which the most severe traffic loadings are generated, and for which the tie and fastening systems must perform effectively.

However, even in the high speed passenger applications in North America, the load environment is more severe than that found elsewhere in the world because of distinctly different operating and maintenance characteristics, that include the operation of heavy axle load freight cars over passenger tracks. This in turn generates the severe load requirements under which fasteners and ties must perform.

In general, fastener characteristics can be divided into several basic categories, representing different aspects of fastener (or railroad) operational requirements. These include: Track Strength and related fastener strength properties, operations and maintenance requirements, and cost/benefit issues. The first of these areas, Track Strength is directly related to the ability of the fastener system to perform under the railroad loading environment into which it is installed. For North American freight applications this usually means severe curvature, heavy axle load operations. The next area, operations and maintenance requirements, relates to the practical considerations that make for effective track systems because they promote ease of application and matching of fastener properties and life with the remainder of the track system. The last area, cost/benefit issues, while outside the scope of this paper, can never be avoided in the private North American railroad

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environment. The benefits that are obtained by the use of a appropriate fastener system must be evaluated in conjunction with their costs. However, it should be noted that life cycle costs of the system must be evaluated, rather than either first costs or isolated component costs.

It is the purpose of this paper to concentrate on the performance requirements of fastener systems in track from the point of view of examining the role of and the requirements for the fastening system. It is not the intention of this paper to define performance specifications or performance tests, but rather to look at the role of the fastener in the railroad track structure and examine its operating environment, and the functions that it serves. It is left to the operating railroads themselves to extend these performance characteristics and to define specifications from these characteristics based on the specific requirements and circumstances of the railroad. Specifications such as those developed by the American Railway Engineering Association (Reference 1), are designed to help guide the railroad in this area of specifications and specification testing.

II TRACK STRENGTH

Track strength refers to the ability of the track structure, and specifically herein, the ability of the fastener system to perform its functions in the railroad operating environment into which it is installed. Thus this area deals with those fastener performance characteristics which affect the strength of the track structure itself and its performance under all loading conditions encountered in the railroad environment; both vehicular and environmental.

The primary function of the fastening system is to secure the rails to the cross-ties. In order to accomplish this the fastenings must be capable of withstanding the loads that are applied to the rail and transmitting these loads to the rest of the track structure without excessive movement of the rails and without failure of the fastenings. As part of this function the fastening system must retain the gage of the track under train operations; i.e. dynamically as well as statically. In addition the fastenings must facilitate the ready removal of the rails from the remainder of the track structure for ease of replacement as well as performance of additional maintenance activites.

For the purpose of this paper, these performance characteristics have been divided into four basic areas: longitudinal restraint, gage widening/rail rollover, lateral shift/track buckling, and vertical dynamics. Each of these areas refers to both a performance area for the track structure as a whole and within it, the performance of the fastening system. It should be noted here, that the fastening system is treated as a part of the overall track structure, and as such its performance characteristics must match those of the other track components and of the system as a whole.

A. LONGITUDINAL RESTRAINT

Longitudinal restraint refers to the ability of the track structure to withstand longitudinally applied loads, without movement or failure. For fastener systems, this refers to the restraint of the rail from movement in the longitudinal direction. Thus for elastic or other nonconventional fastener systems, the fasteners may replace the traditional wood tie anchors which perform the same function.

Longitudinal forces are induced both mechanically, by train action and thermally, by changes in ambient (and hence rail) temperature.Mechanically induced forces are developed by the acceleration or tractive effort of the train or by decelerating and braking of the train.While forces as high as 60,000 lbs per rail have been measured, the mechanically induced forces generally are less than 20,000 lbs per rail(Reference 2).Additional rail creep is also generated by the action of passing trains.When this creep is restrained by "hard spots" in the track such as turnouts or crossings,significant longitudinal compressive forces can be built up.(Reference 3) The fastenings must restrain these forces to avoid their transmission into these key track structure points.

Thermally induced longitudinal rail forces can be either compressive or tensile in nature, depending on whether the temperature is above or below the rail laying (or force free) temperature. Figure 1 shows the relationship between rail force and temperature change: note the significant changes in force that occur with temperature.Figure 2 shows the distribution of the axial force along a length of tangent track. This force distribution consists of the two end or "breathing" zones in which longitudinal movement of the rail can take place and a central "constrained" zone in which no longitudinal movement takes place. It is generally in the "breathing" zones, which are the ends of the CWR strings, that most of the longitudinal movement takes place and for which it is most critical to have good longitudinal restraint characterisitics in the fastener.For a more complete description of this phenomenon, the reader is referred to Reference 4.

In the case of curved track the presence of large longitudinal forces can result in the movement of the curve itself. This is due to the curve attempting to readjust it s force free temperature by either moving in (at low temperatures) or moving out (at high temperatures). This curve readjustment causes, in turn, longitudinal movement in the track which must be restrained by the fastening system.

When the temperature decreases, and a longitudinal tensile force occurs in the rail,the possibility of a track pull-apart arises. In the event of a pull-apart,the fasteners act to restrain the ends of the pull-apart from separating and one performance criterion for longitudinal restraint has been a maximum pullapart gap. Table 1 presents calculated longitudinal restraint requirements for rail gaps of .5 to 2.0 inches. Thus in order to limit a rail pull-apart to a one inch gap under a temperature change of 75 degrees F ,a minimum restraint of 1800 lbs per assembly would be required.

It should be noted that the largest temperature variations occur between maximum (and minimum) annual temperatures and the laying or force free temperature. This is therefore seen in track gradually over the course of a year. More frequently the fasteners encounter a daily temperature variation which can result in significant temperature variations several hundred times in a year.

For conventional tie in ballast track,the longitudinal restraint of the fastener does not have to be greater than the longitudinal restraint of the tie in the ballast. In fact, significantly greater restraint beyond the point at which the ties "plow" in the ballast would be unnecessary. Table 2 presents the results of a large number of longitudinal resistance tests for wood cross ties in ballast taken on four major U.S. freight railroads (Reference 5). It should be noted that the 'average " value of the test data shows that it takes about 2180 lbs to displace a cross-tie longitudinally .08 inches. This value is the point of definition for longitudinal restraint in Reference

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5). The largest longitudinal resistance measured on any cross-tie was 4900 lbs. These values correspond to an "average" fastener longitudinal restraint of 1590 lbs and a maximum fastener longitudinal restraint of 2450 lbs. It is worth comparing these values to the AREA concrete tie fastener recommended value of 2400 lbs(1) and the AMTRAK recommended value of 3000 lbs(7).

Finally, it should be noted that the longitudinal restraint performance of the fasteners must be obtained in an active environment, i.e. under the passage of trains. In order to simulate this environment, some railroads have carried out longitudinal restraint testing in the presence of a dynamic excitation, such as that produced by a vibratory motor mounted on the rail(8). While this does not change the longitudinal restraint requirements for in-track performance of the fastener, this type of test has been used to more closely simulate the effect of train dynamics on the longitudinal restraint performance of the fastening system.

B. GAGE WIDENING/RAIL ROLLOVER

Gage widening represents one aspect of the lateral restraint characteristics of the fastener system. Since one of the key functions of a fastener system is to maintain track gauge, the ability of the fastening system to withstand applied lateral loads without excessive lateral movement and hence gauge widening is of critical importance.

Gage widening can be defined as any increase in the standard track gauge of 4' 8 1/2", measured at the gauge point, 5/8" below the top of the rail head(1). Gauge widening can be caused by any combination of three factors:

(1) Rail Wear: abrasive wear on the gauge face of the rail head, usually on the high rail of curves.

(2)Rail Translation: lateral displacement of the base of the rail relative to the tie.

(3) Rail Roll or Rotation (Reference 9): Any rotation of the rail section from the "original" vertical axis of the rail and the corresponding lateral displacement of the rail head with respect to the rail base (Figure 3).

Rail wear, which is normally associated with the rail and load environment ,can be indirectly affected by the fastener system. This has been reported recently in Rail International (Reference 10) where a group of East European researchers presented the results of a study of the effect of fastener lateral stiffness on rail gauge face wear. These results, which are presented in Figure 4 show that for concrete tie-fastener systems,the use of rigid fastening systems can cause significantly higher rail wear than the use of resilient elastic fastenings.Similar behaviour can be extended to wood tie track, however in that case it should be noted that excessively soft lateral stiffness, i.e. as can occur in spike killed ties, can also result in abnormal rail wear behaviour.

Rail translation has relatively small effect on gage widening in the case where good tie plate-tie connections exist, as in the case of newly spiked track. This is clearly seen in Figure 5, taken from AAR test data (14), where for "new" track conditions, the rail translation is less than 10% of the total rail head deflection (0.05 inches). However, as the spike-tie system deteriorates with age and traffic, the magnitude of the rail translation increases. Thus for ties in "poorer" condition, the magnitude of the rail translation can be 0.25 inches under relatively low loads (Figure 6, Reference 16) and more than 0.5 inches under higher loads (17). In fact, for FRA Class 1 type track, it appears that rail base translation can represent a significant percentage of rail head displacements, as high as 70% for some cases (17).

However, it is generally rail rotation that causes the largest additional gauge widening in an environment of high lateral and longitudinal loadings, such as that found in heavy curvature, severe grade heavy haul railroad territory. In this type of environment, dynamic gauge widening, i.e. the instantaneous increase in gage caused by dynamic lateral loads due to traffic, must be controlled so that the gauge does not increase sufficiently for a wheel to drop in and cause rail overturning and consequently a derailment. As can be seen in Figure 7, in a worst case situation of flange worn wheels, chipped tread, and gage face wear (all within allowable AAR interchange limits for freight cars) a gauge widening of 1.32 inches (exclusive of rail wear) will result in wheel drop in. Noting that 0.25 to 0.50 inches of this can be attributed to rail base translation and/or improper gaging during rail laying, then it can be seen that an additional dynamic gauge widening, such as that due to rail rotation, of about one inch could be potentially catastrophic.

It should be noted that rail rotation, as seen in Figure 3, occurs under combined lateral and vertical loading. When the vector sum of the lateral and vertical loads is such that it falls outside the base of the rail, as noted by the dashed line in Figure 3, rail rotation takes place, and is resisted at this point by the fastener. For 132 RE rail, this occurs when the ratio of lateral to vertical loads (the L/V ratio) is greater than 0.65.

In severe traffic environments, dynamic lateral wheel loads as high as 40,000 lbs have been reported and loads in the 20-25,000 lb range have been measured under a variety of operational conditions (11). These loads, however, represent very low probability of occurrence events in the railroad environment. Other attempts to statistically define the load environment have presented lateral/vertical loads combinations to be 39/43.2 kips as a maximum loading combination and 9.7/16.2 kips as a "frequent" loading combination (12).

Laboratory investigation of the gauge widening strength of wood tie and different fasteners reported that it takes between 10,400 and 31,500 lbs of lateral load (with no vertical load) to widen track gauge one inch (13). Figure 8 presents the results of laboratory tests of wood ties with cut spikes that show the lateral and vertical load combinations that can cause 1.00 inch track gauge widening (14). It should be noted that the maximum load combinations referred to above, are sufficient to cause one inch gauge widening in conventional wood tie track. Similar lateral and vertical load combinations have been found that will cause two inches of dynamic gage widening in "weakened" wood ties with cut spike fasteners. (15)

Finally, it should be noted that the presence of longitudinal loading can also contribute to rail rollover, particularly in a curve where a lateral component is present. This becomes quite significant under thermal loading, both in compressive loading (from temperature increases) and in tensile loading (from temperature decreases) where a significant contribution to rail rotation forces occurs.

C. LATERAL SHIFT/TRACK BUCKLING

Lateral Shift and lateral track buckling represent the second aspect of the lateral restraint characteristics of the track structure. Although these aspects are primarily functions of the tie and ballast , the fastening system does have a secondary effect on the track behavior in this area.

However, even though the fastener performance effect on lateral shift is secondary to the tie/ballast interaction effect, it still exists and it is worth a brief mention.

In the lateral plane, the track structure can be considered a frame, with the rails acting as longitudinal elements and the cross-ties acting as transverse elements. Within this frame, the fasteners can be considered as springs, so that variations in fastener torsional rigidity, i.e. the equivalent fastener stiffness in the plane of the track structure, will change the overall lateral strength of the track itself. The analysis for this type of track behavior is presented in Reference 18. Using that analyses and substituting fastener torsional resistance values presented in Reference 19, the lateral deformation of a rail-tie structure (no ballast) was calculated for cut spike type and elastic type fasteners. The results are presented in Figure 7. It can be clearly seen in this Figure that the lateral deformation of the rail-tie structure was significantly reduced with an elastic type fastening system. Thus, noting the dominant effect of the tie/ballast interaction on this behavior, it can be observed that good torsional fastener resistance will further strengthen the track laterally and reduce lateral deformation behavior.

In addition, the fastening system acts to reduce lateral shift under those circumstances in which the curves tend to move in or out laterally due to large temperature variations and the consequential readjustment of the track. In these circumstances, both the longitudinal restraint characteristics of the fastener, as well as the torsional resistance characteristics, come into play in maintaining the integrity of the track.

D. VERTICAL DYNAMICS

The fourth and final characteristic area to be considered under the definition of track strength is the response of the track structure and hence the fastening system to vertical loads. The fastening system must transmit the vertical loads applied to the railhead to the cross-ties and subsequently to the ballast and subgrade below. Since the railroad environment is a dynamic one, as noted earlier, it is the dynamic loads that must be distributed and transmitted to the rest of the track structure.

The vertical load environment of both freight and high speed passenger traffic is one that has been extensively characterized through field testing. Figure 10 presents one such set of dynamic vertical loads for both freight and passenger traffic (20). It should be noted that at the 0.1 percent level (one wheel out of every one thousand wheels) the freight traffic applied a vertical load of over 50,000 lbs. Since the static vertical wheel load for freight equipment is no more than 33,000 lbs and the passenger wheel load considerably less, it is apparent that the dynamic augment to the static wheel loading is quite significant and must be considered in the performance characteristics of the fasteners.

It is worth noting here that a major source of the dynamic augment to the static vertical load is the presence of wheel flats (up to 2 1/2 inch flats are allowable under AAR interchange rules) or other wheel anamolies on the equipment. These wheel loads can provide dynamic impact factors of two or more ,as can be seen in Figure 11, which presents AAR test data on wheel flat effects (21). In addition, defects on the surface of the rail head, such as engine burns, corrugations, high or low welds, and rail joints, can result in similar dynamic impact effects.

Since the fastening system can provide resilience in the vertical direction in both the up and down directions (the double elastic fastening system) the performance of the fastener during rail uplift under traffic must be considered. Thus it is necessary for the fastener system to have sufficient vertical restraint(or toe load) to support the rail- tie system during uplift, which can occur under vertical wheel loadings of more than 25,000 lbs (Reference 22).Under these circumstances, each fastening pair must support the weight of the cross-tie and rail section without excessive deformation of the fastener and without failure of the fastener components.

III OPERATIONS AND MAINTENANCE

In addition to the track strength performance requirements discussed in the previous section, there are operational and maintainability requirements that enter into any set of performance characteristics for fastener systems to be used in the railroad environment. These are the "practical " considerations that make for an effective and easy to use fastener system.

This class of fastener characteristics can be as important as the track strength (performance under load) characteristics discussed previously because they address real issues of concern to the maintenance personnel regarding the use of the system in the field environment.

These characteristics include fastener life and fastener maintainability. This section will address these characteristics from the point of view of the end user and the job which must be performed, i.e. performance characteristics, rather than from any specification or individual fastener features point of view.

A. LIFE

Fastener life, simply defined, refers to the period of time or cumulative tonnage until the fastener, or its individual components,must be replaced.

It can be argued that selection of a "suitable" fastener life is an economic criterion, rather than a true performance criterion, however it obviously falls into that category of "practical " issues that must be addressed in the railroad environment.

The life of the fastener and its components is very significantly effected by the operational environments that were described in the previous section.

Another consideration in fastener life characteristics is that the fastening system must exhibit its required performance characteristics in situations which may call for its repeated removal and reassembly in the field, as is the case when the rail is transposed or replaced frequently on sharp curves in heavy tonnage territory or when frequent destressing is carried out. This leads to a requirement for the reassembly and reuse of the fastener components without loss of performance (as defined previously in track strength). Component failure can therefore be defined to be the point where the fastener s performance characteristics drop below appropriately defined levels, or when the component physically fails and is unable to carry out it's functions. Thus, clip failure can be defined to occur either when the clip fractures or when it loses enough "toe-load" as to be unable to maintain the defined level of longitudinal restraint.

A commonly used fastener life standard is that life equal to the life of rail in tangent track. Under heavy tonnage types of operations, this can be 500 MGT of cumulative traffic or 25 years at 20 MGT annual traffic. Under 100 Ton car traffic, this translates to over 15 million loading cycles (axles) for the fastening system.

B. MAINTAINABILITY

Maintainability, as discussed in this section, refers to those characteristics of the fastening system (as opposed to individual fastener features) that provide for ease of use and operation in the field.

Thus,an important characteristic in this area is the capability of being easily installed and removed by local maintenance forces with a minimum (or no) specialized insertion and removal tools,preferably with regular hand tools. This should be performable by the general maintenance of way worker and should be readily integratable with other maintenance activities.This feature is extremely important from the point of view of enabling immediate response by local maintenance forces to spot or local problems.

In conjunction with this, the fastener system should require only a minimum of adjustment, so that proper installation to achieve the required performance is "automatically" obtained and improper installation minimized.This should also minimize the need for periodic readjustment of the fastener system in track.

In addition, the fastener system should be capable of mechanized installation and removal, in conjunction with large maintenance operations. This can result in significant labor and productivity savings, which while not a performance characteristic,is an important practical feature.

Another "feature" which is of real practical significance is the ability to see and rapidly inspect as many of the fastener components as is practical. This is invaluable in facilitating regular inspection of the fastener in conjunction with other routine inspections and activities.

Finally, another consideration is the ability to resist catastrophic failure, i.e. survivability and mimimum performance in the event of a derailment.

As noted earlier, these characteristics, while not necessarily being considered as true performance characteristics,do address fundamental issues of concern to the maintenace personnel who must work with the track, and hence the fastener system.

IV COSTS AND BENEFITS

The final category of fastener characteristics is one that addresses not just the physical performance of the system in track, but also the overall cost of the track system. This issue is of real concern to the private freight railroads of North America in that they must operate in an environment that minimizes cost and maximizes measurable benefits.Thus in the

final analysis any benefits that are obtained by the use of an appropriate fastener system must be evaluated in conjunction with their costs, which in the railroad environment must be life cycle costs.

It is beyond the scope of this paper to address the issue of costs and benefits. Rather it is sufficient to mention this is as an important issue, which can be considered a "performance" issue when the railroad environment is viewed as a cost center.However, for more detailed investigations into the costs and benefits of tie fastener systems, the reader is referred to References 23 and 24.

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TABLE 1 — Fastener Longitudinal Restraint for Pull a Part

RAIL BREAK "GAP" (INCHES)	RESTRAINED LENGTH OF TRACK ON EACH SIDE OF GAP (FT)	TIE SPACING (INCHES)	LONGITUDINAL RESTRAINT PER FASTENER (LBS)
0.5	86	19.5	3627
0.75	129	19.5	2720
1.0	172	19.5	1814
1.5	258	19.5	1360
2.0	344	19.5	907

RAIL WEIGHT = 132 RE

MAXIMUM TEMPERATURE CHANGE = 75° F

TABLE 2 — Longitudinal Resistance of Wood Ties in Ballast

LONGITUDINAL RESISTANCE* (LBS PER TIE)

	UNCONSOLIDATED **			CONSOLIDATED **	
MGT	TANGENT	CURVE		TANGENT	CURVE
0	2588	2763		3049	2712
.5 to 1	3030	3476		3279	3279
1 to 2	3194	3814		3530	3296
Maximum Individual Case			4900		
Average (all cases			3180		

* Defined to be longitudinal force necessary to displace tie .08 inches (Reference 5)

** Average of test data from Boston & Maine, Southern, St. Louis-South Western, and Missouri Pacific Railroads

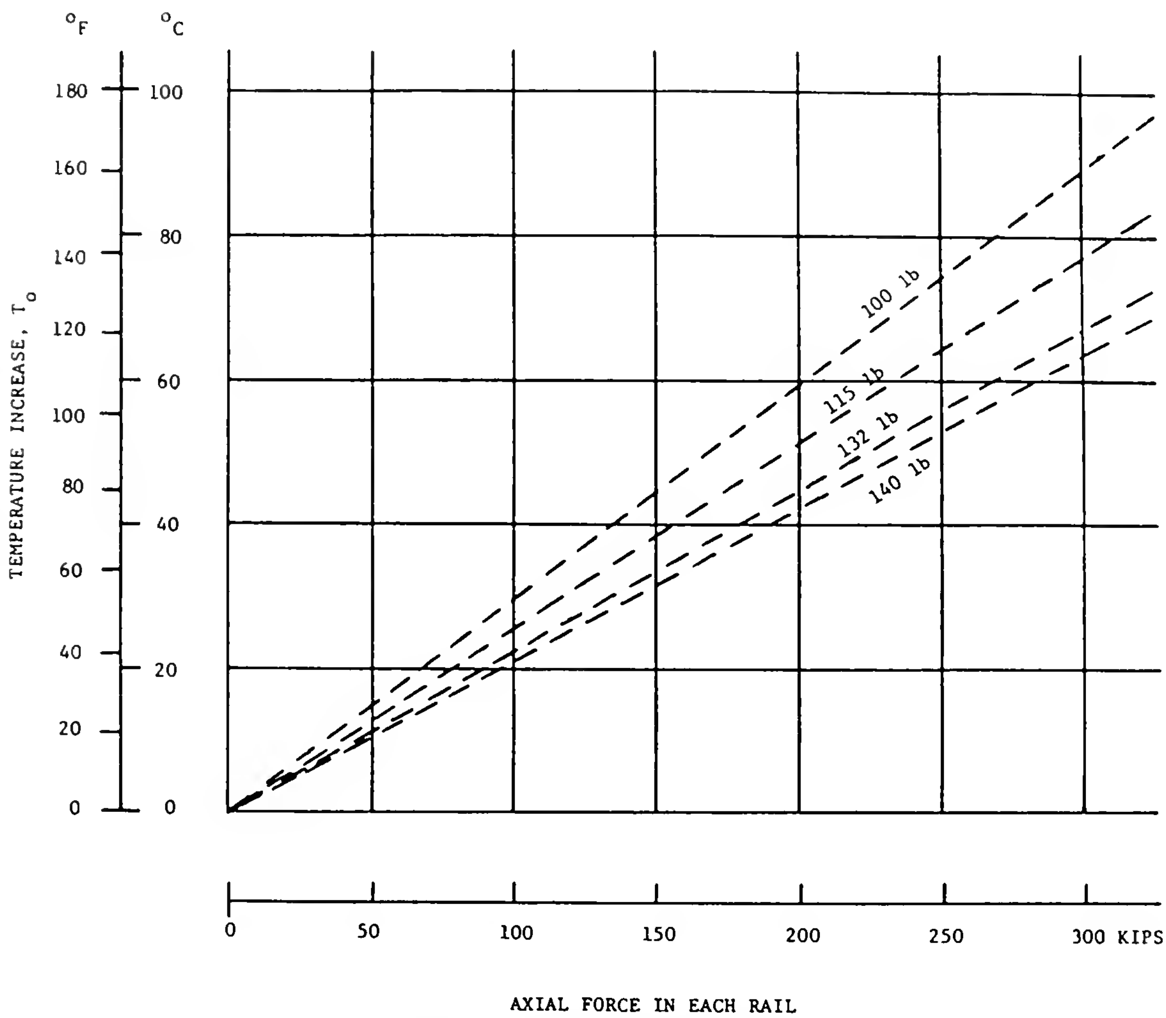


FIG. 1. Rail Temperature Increase vs Axial Force in Rail

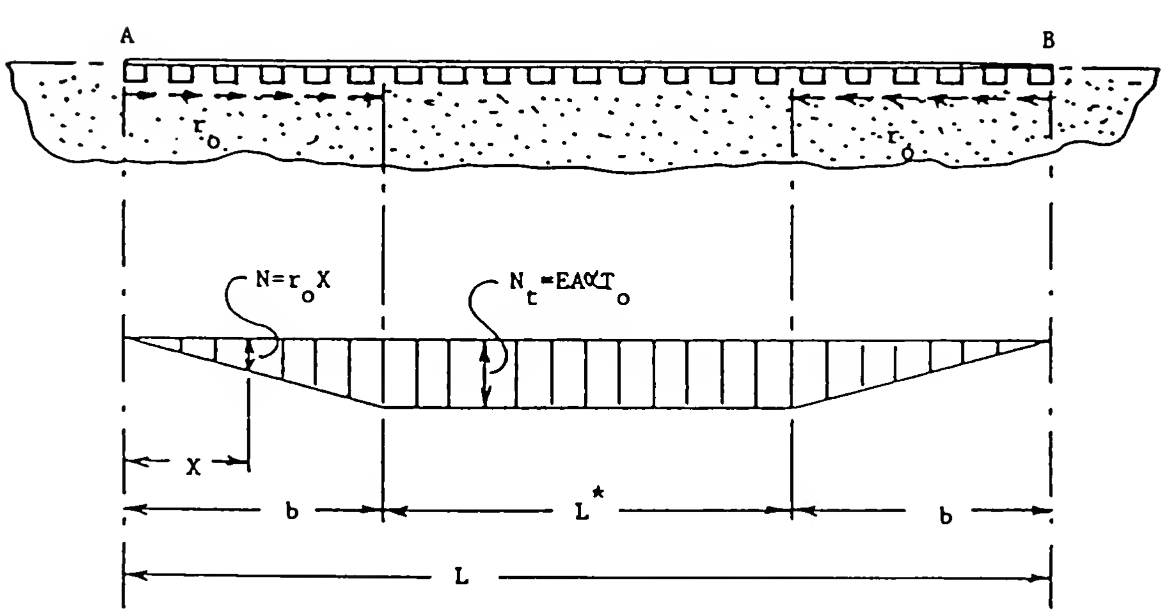


FIG. 2. Axial Force Distribution in a Track of Length L

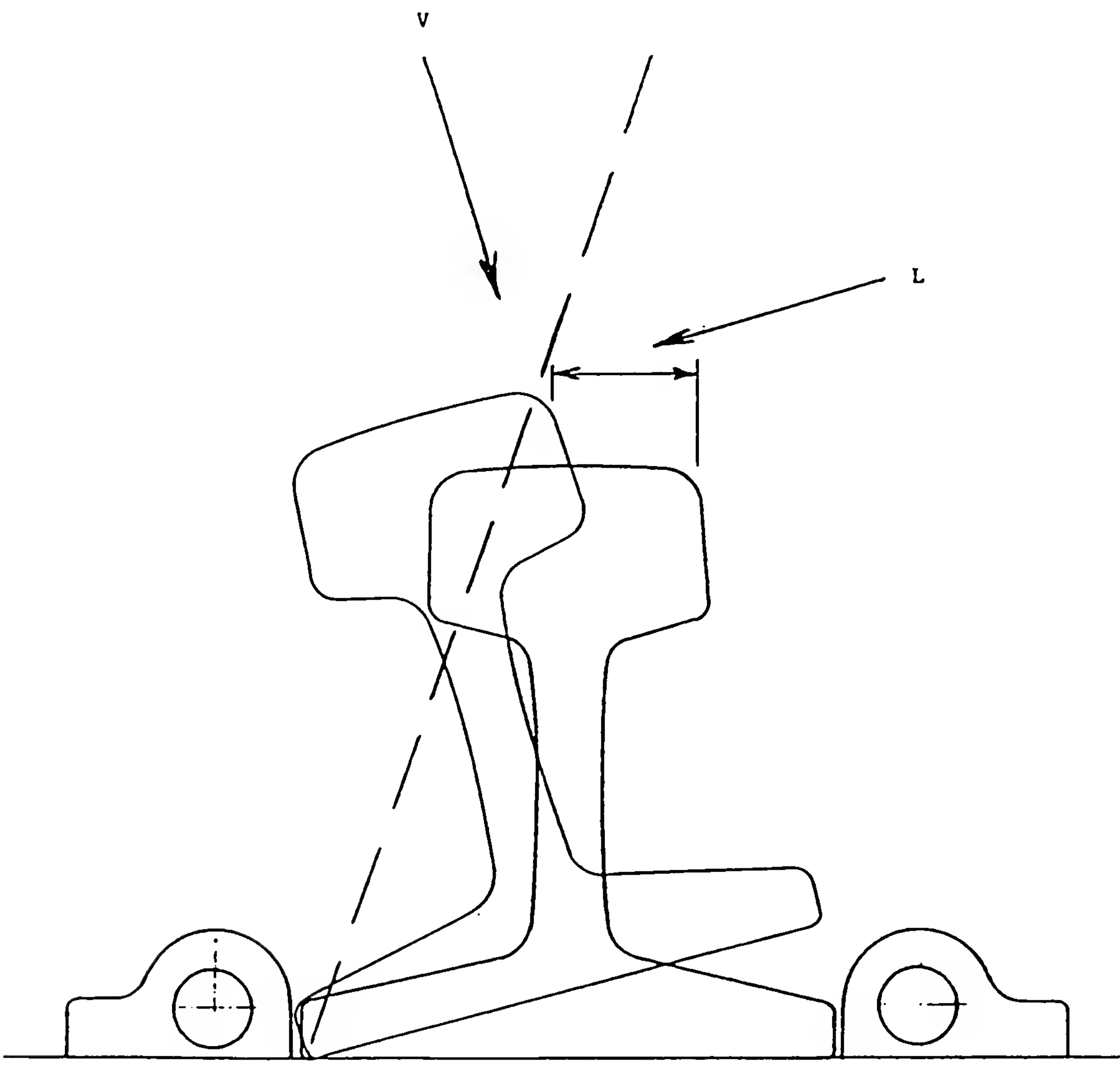


FIG. 3. Rail Rotation Under Lateral and Vertical Loading

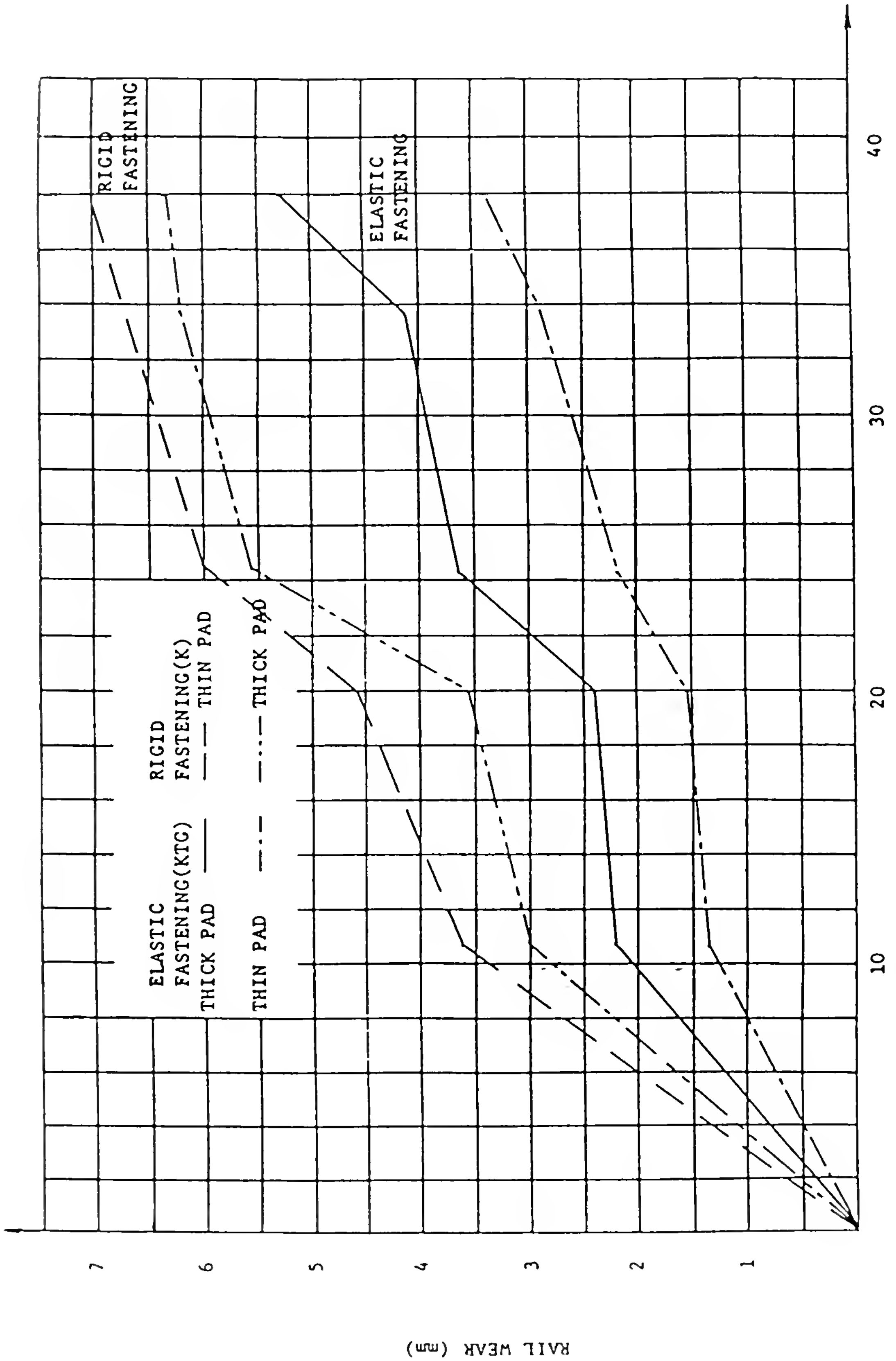


FIG. 4. Rail Wear for Rigid vs Elastic Fastenings

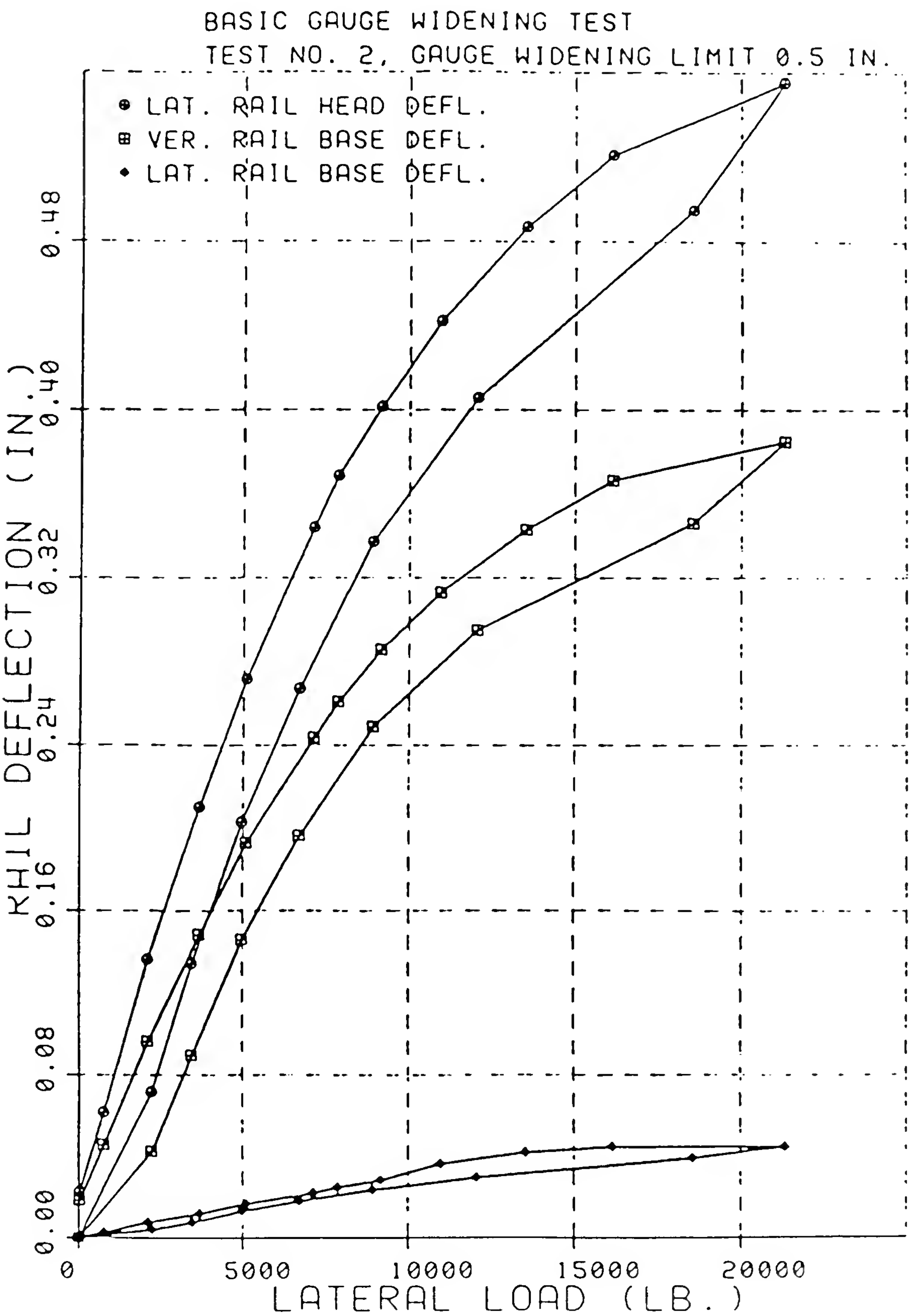


FIG. 5. Graph Showing Various Rail Deflections vs Lateral Loads, for Zero Vertical Load

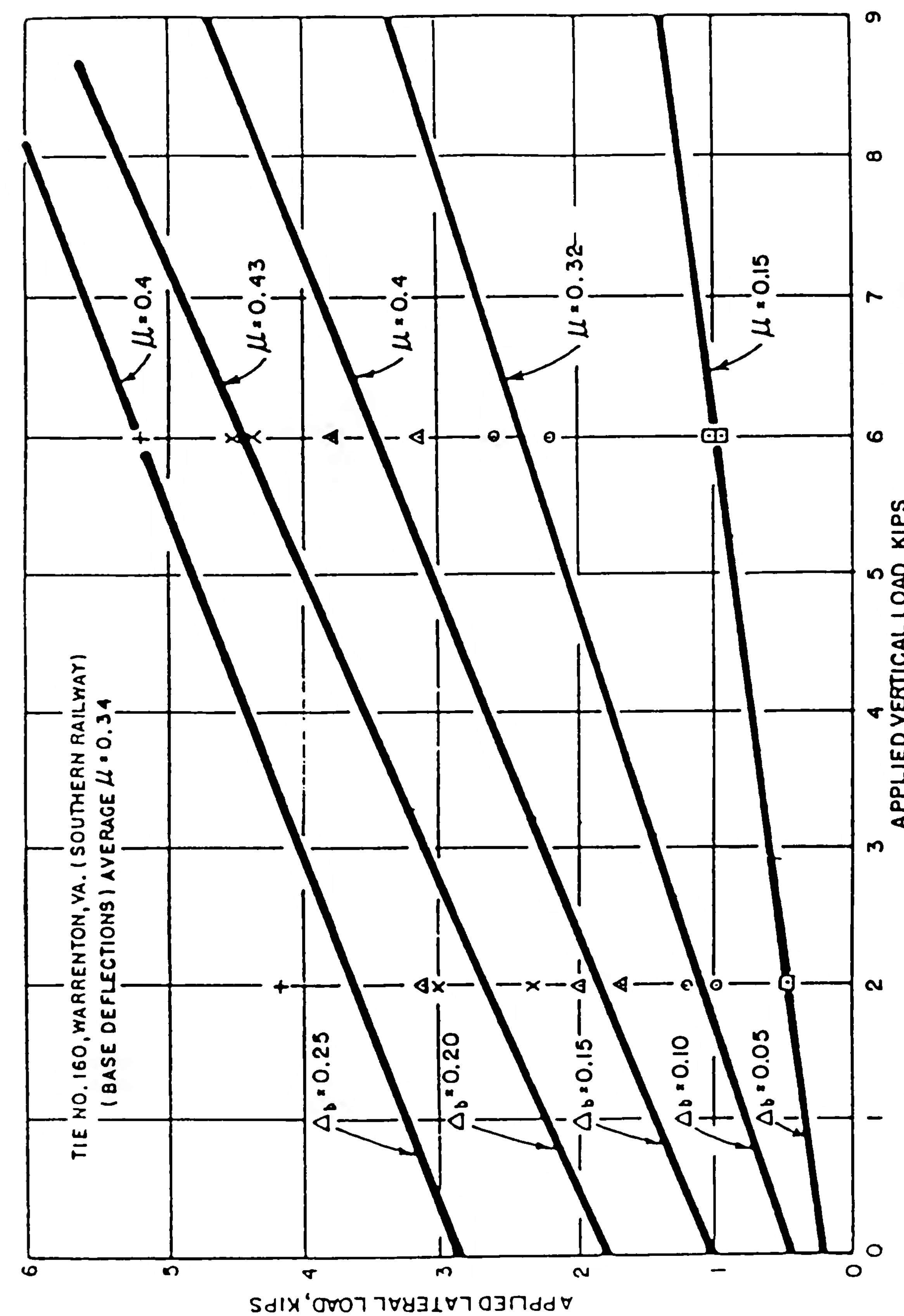
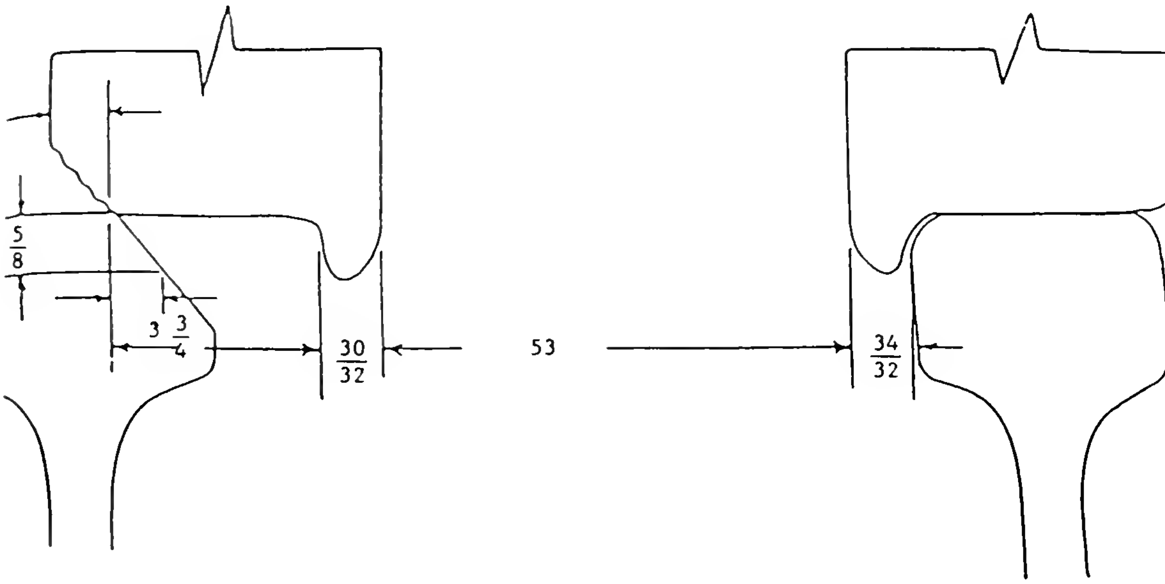


FIG. 6. Data from Southern Railway Tests at Warrenton, Va. Rail Base Deflections at Tie No. 160



THICKNESS SUM OF 2 WORN FLANGES *	$\frac{64}{32}$
BACK TO BACK OF WHEEL FLANGES (MIN)	+53
AVAILABLE TREAD CHIPPED WHEEL**	+ 3 $\frac{21}{32}$
	58 $\frac{21}{32}$
GAGE WEAR AT TOP OF RAIL	$\frac{27}{32}$
RANGE OF MAXIMUM DYNAMIC GAGE SHORT OF DERAILMENT	57 $\frac{26}{32}$
NOMINAL GAGE	56 $\frac{16}{32}$
ALLOWABLE DYNAMIC GAGE WIDENING	1 $\frac{10}{32}$ (1.32) INCHES

* Assume that opposite wheel flange has half the allowable wear surface

** Chipped rim is unusual under normal conditions but it could exist under AAR Rule 41

FIG. 7. Maximum Track Gage—Worn Wheel Flanges and Chipped Rim Wheel on Curve Worn Rail

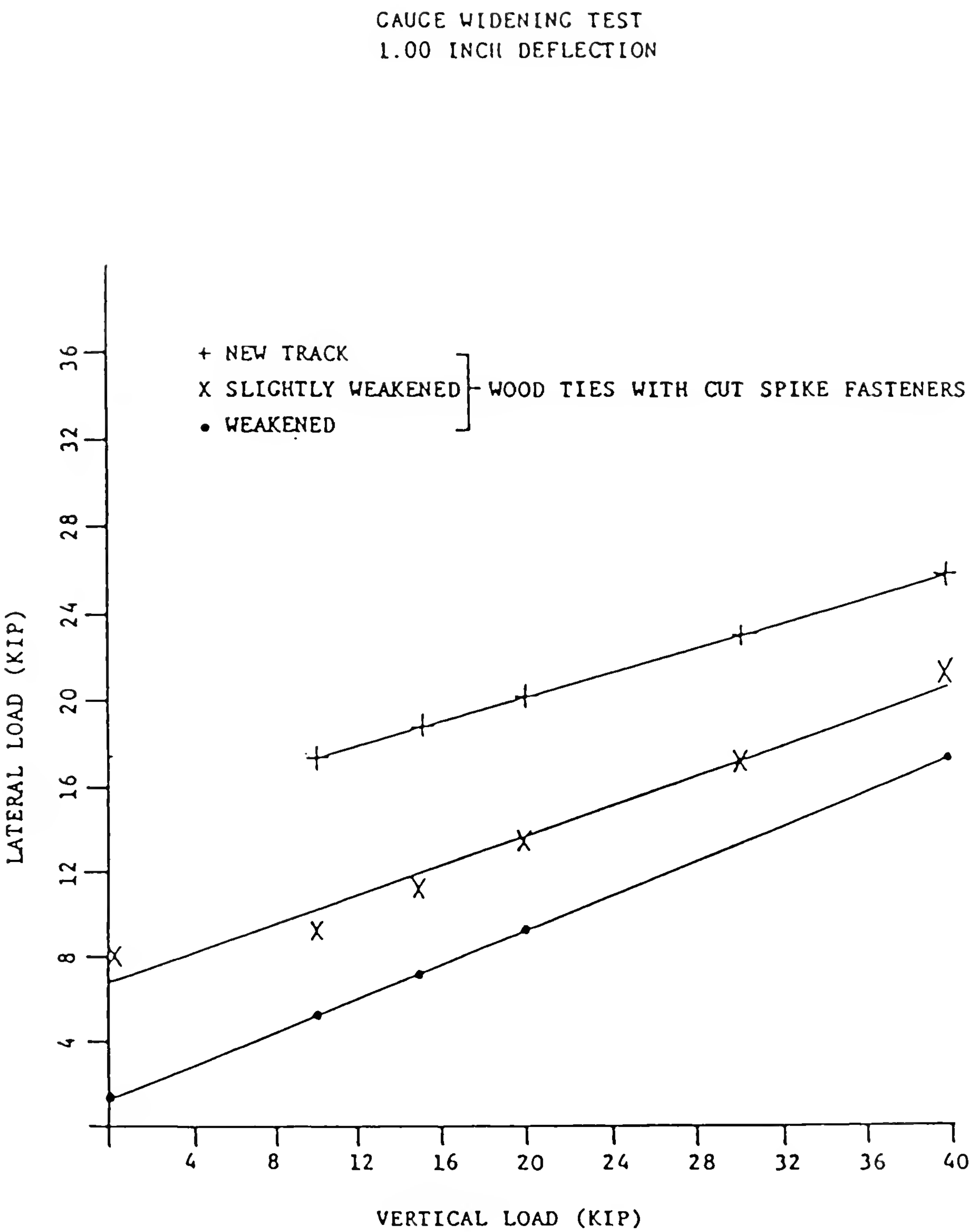


FIG. 8 Vertical-Lateral Load Combinations For 1.00 Inch Gauge Widening in Wood Tie Track

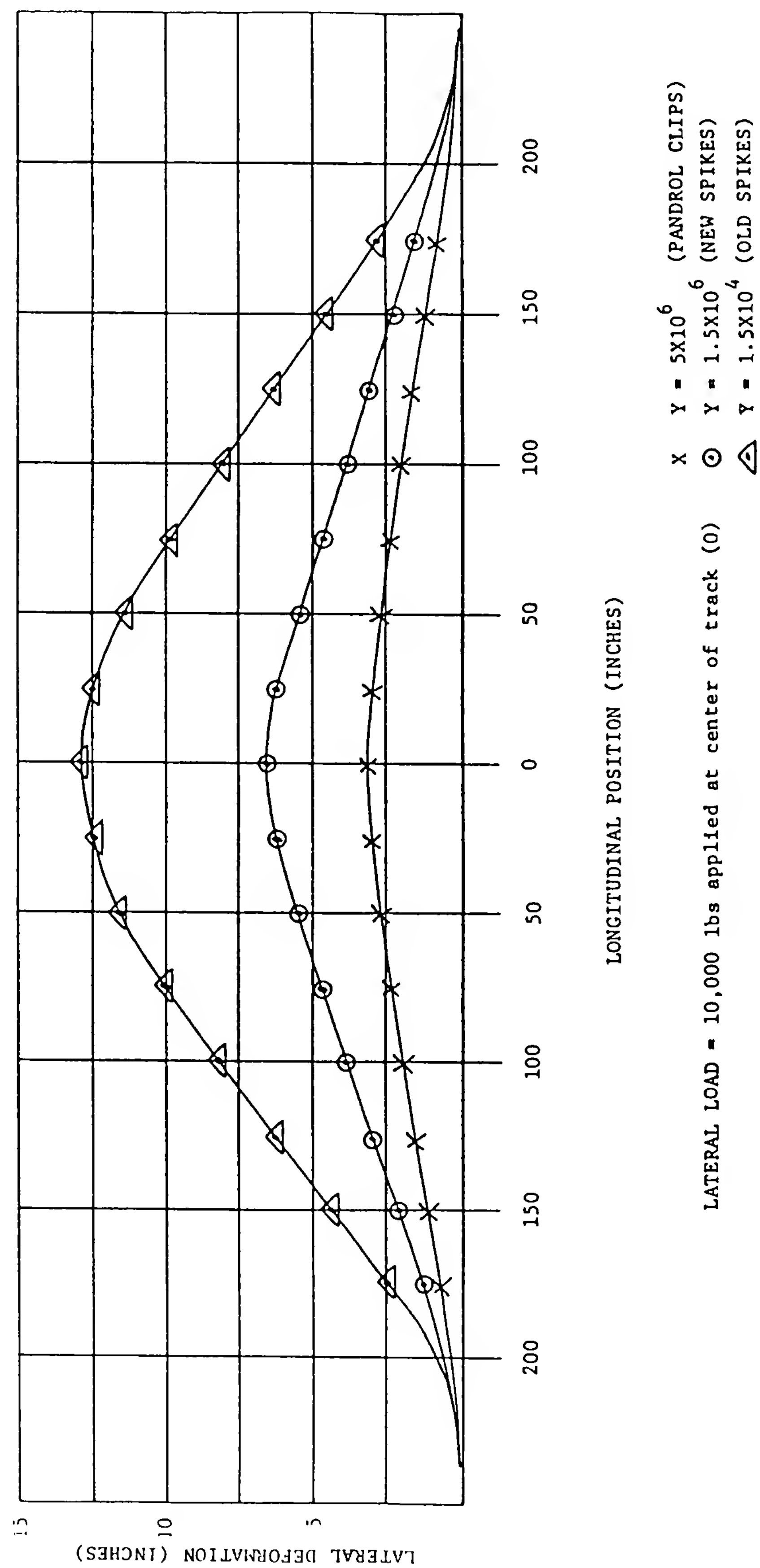


FIG. 9. Lateral Deformation of Track (No Ballast) Effect of Fastener Stiffness

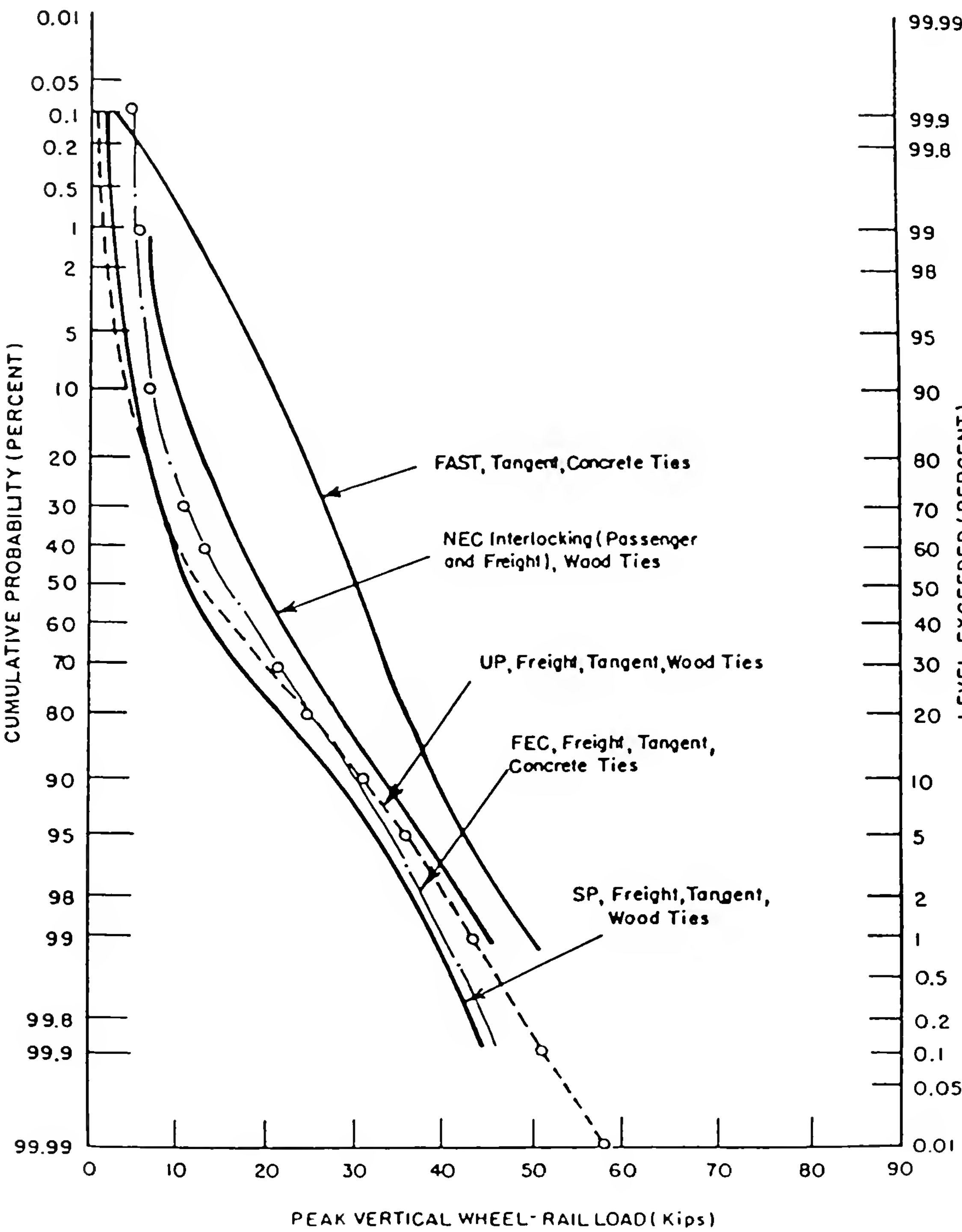
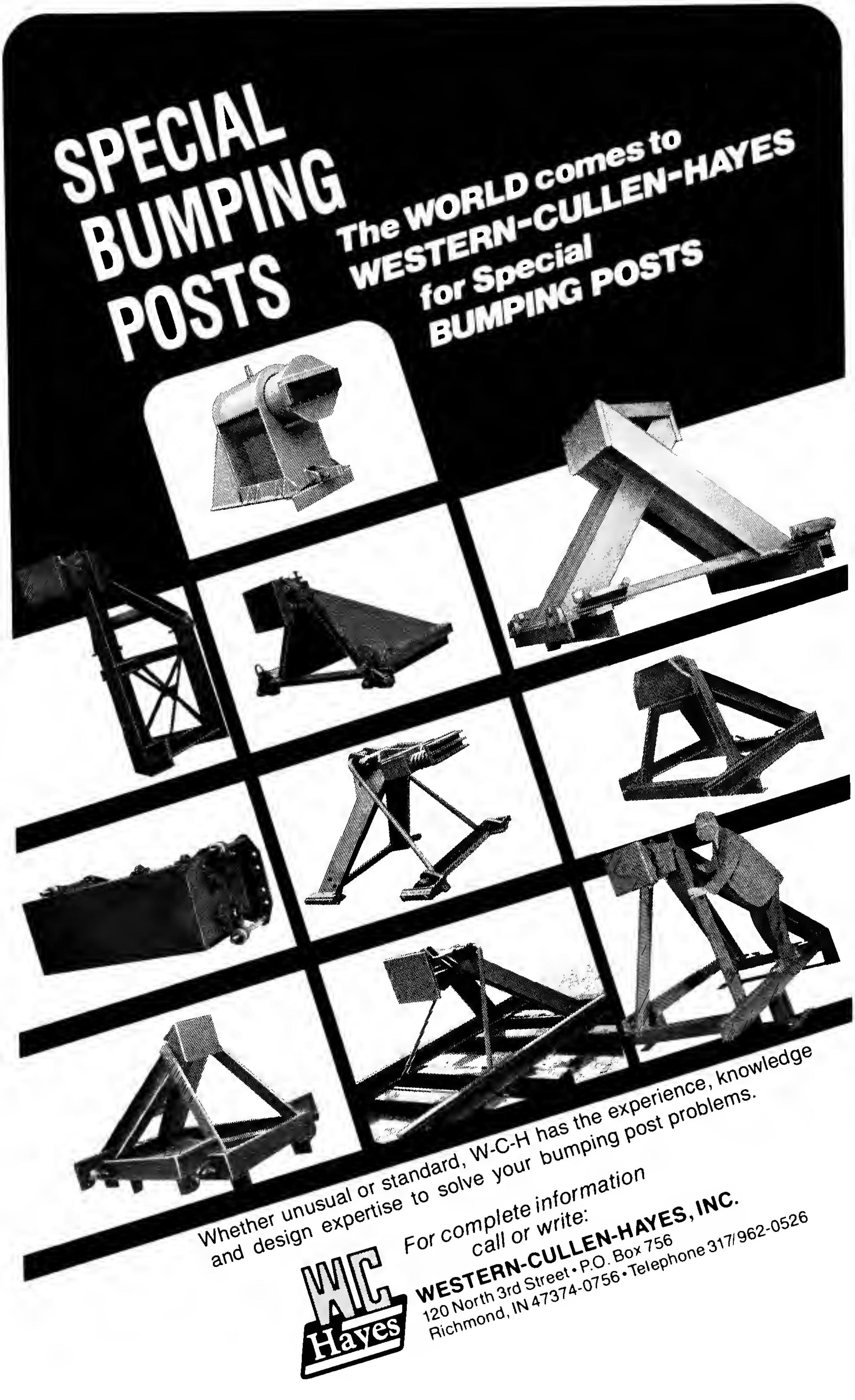


FIG. 10. Wheel-Rail Load Spectra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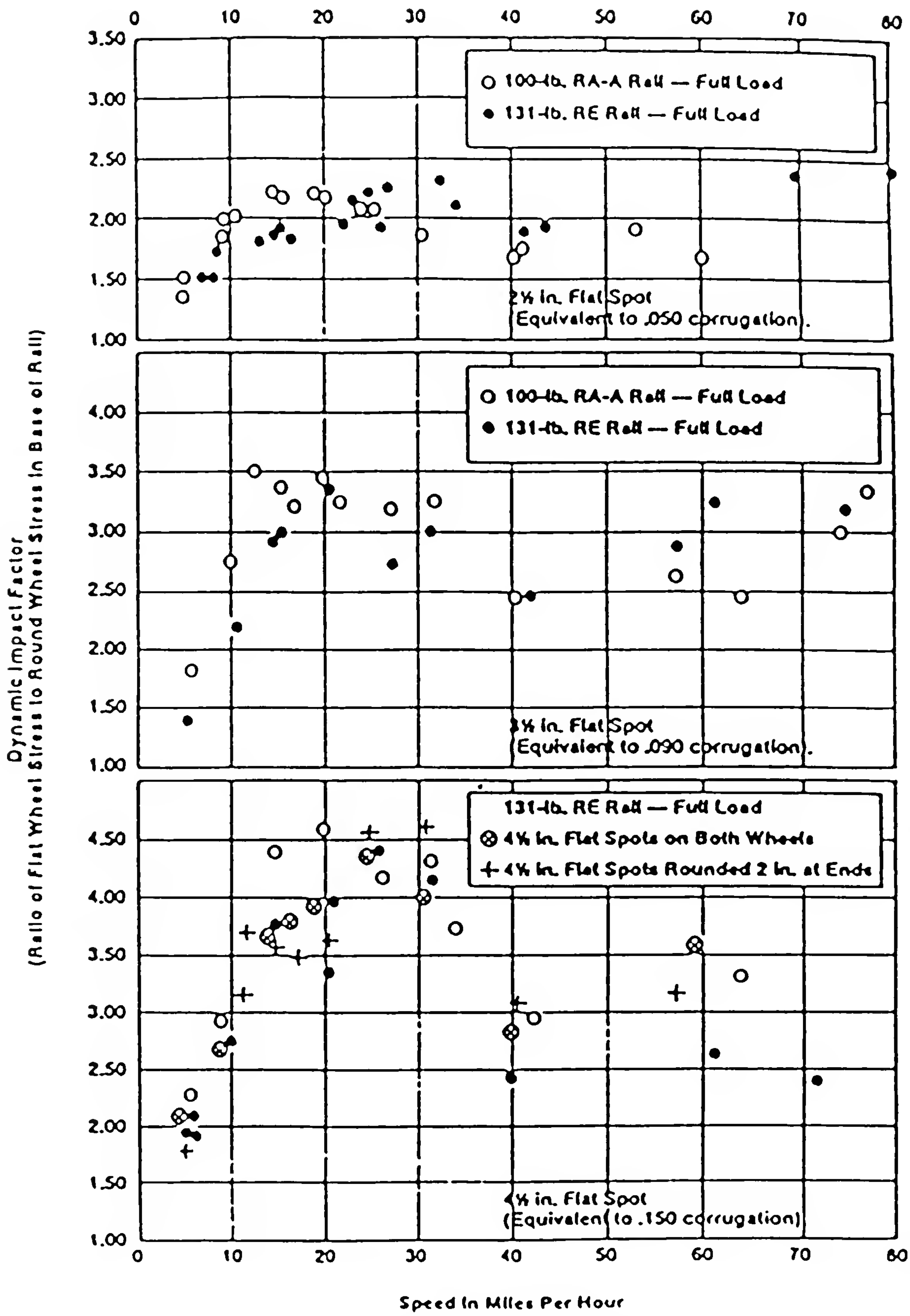

FIG. 11. Ratio of Flat Wheel Stress to Round Wheel Stress in Base of Rail at Various Speed. Fully Loaded Car, 100 lb. and 131 lb. Rails.

COMMITTEE 22—ECONOMICS OF RAILWAY CONSTRUCTION AND MAINTENANCE

Report of Subcommittee No. 8

"Economics of Rail Surface Maintenance"

C.D. Barton (Chairman—Subcommittee No. 8), W.H. George, D.E. Turney, R.J. Begier, R.G. Brohough, H.R. Davis, H.B. Durrant, J.T. Hunter, F.S. Mitchell, E.J. Rewucki

The growth of traffic volume, including increased use of heavy (100 ton) cars in unit trains, combined with higher operating speeds continues to generate increased track maintenance problems, not the least of which is rail surface maintenance. Obviously, it is unrealistic to suggest a wholesale reduction in car size or train speed to deal with this situation; therefore, it is essential that ways be found to reduce the problem of rail corrugation, batter and surface wear under existing and anticipated service conditions and thereby extend the life of rail in the track. Obviously, any improved maintenance techniques that will increase the service life of rail will impact favorably on all aspects of track maintenance.

To that end, a questionnaire was distributed to all Class I Railroads in an effort to assemble available data on the subject of rail surface maintenance and explore the possibility of formulating recommendations on the subject. Twelve Railroads responded which have a total of 97,962 miles of track throughout North America and represent a wide range of maximum gradient and curvature. In addition to traffic volume, the alignment and gradient of a track will greatly influence the problems associated with rail maintenance.

In the development of recommendations relative to rail maintenance, two major areas must be considered. First, the rail to be used when track is constructed new or is to be relaid under a renewal program; and second, maintenance practices which are available to prolong the life of existing rail in the track.

The use of continuous welded rail in main track relays as well as yard tracks and sidings has become standard practice in the industry. However, data furnished by the survey indicate that the life of standard carbon steel rail, used for such relays, will be reduced by 25 percent when the number of 100 ton cars is greater than 25 percent of the total traffic. This is true on both curved or tangent track. Under any traffic conditions, the combination of heavy gradients and curvature will also have an adverse effect on rail wear. Available information indicates that "Heat Treated" or special alloy rail has a significantly higher gross ton life than standard carbon rail and, although it is more expensive, there are economical advantages to using these special rails in certain situations.

Special attention should be given to the maintenance of good tie condition and line and surface conditions in areas subject to heavy rail wear. Other than this, the use of rail lubricators at selected locations will help offset the development of rail wear. In spite of all of these preventive measures, corrugations, engine burns and battered joints do develop. It is possible to extend the life of the rail through reconditioning methods such as grinding or building up rail ends by welding. Failure to take corrective action produces a constantly accelerating destruction of the rail, ties and ballast to the point that speed must be either restricted or a rail replacement program must be initiated sooner than would otherwise be necessary.

The criterion for grinding rail, building up rail ends, etc., on most railroads is based on recommendation of responsible Maintenance Officers and available money. This results in a practical criterion but provides no record of deferment. Only five of the Railroads that responded to the questionnaire have specifications calling for grinding when corrugations or other defects result in measurable surface variations. These range from 0.015 inches to a maximum of 0.050 inches. It may be practical on a cost basis to grind out deeper corrugations in the rail in some cases.

The questionnaires indicated that build up of battered rail ends is done on an "as needed" basis and whenever the replacement of jointed rail with welded rail must be deferred.

The Committee offers its sincere thanks to Railroads and their Officers who furnished the available data by completing and returning the questionnaire. It was from that data and reports from the Transportation Test Center and companies engaged in reconditioning rail that the following recommendations were assembled for your consideration.

1. Eliminate the problems associated with bolted joints through increased use of continuous welded rail.
2. Install heat treated or special alloy rail on all curves of 1 degree 30 minutes or greater when the gradient exceeds 0.5% and on all curves of 2 degrees 30 minutes or greater when the gradient is less than 0.5% with annual tonnage greater than 10 MGT.
3. Provide rail lubrication on all curves of 2 degrees 30 minutes or greater and any curve that shows excessive wear on the high rail.
4. Grind rail when corrugations are between 0.010 and 0.040 inches in depth on 30% of the length of rail in curve.
5. Build up joints by welding on a spot basis as required for rail not scheduled for replacement when batter reaches .030 inches.
6. Weld engine burns on concrete ties to prevent vibration or impact damage to the ties.

Representative cost of grinding rail is 1.3¢ per track foot per thousandths of an inch of metal removed. On this basis, the cost to grind to a depth of 0.050 inch over one mile is $3432. The frequency, depth of corrugations and length of track to be included in a rail grinding program is, of course, highly variable subject to traffic levels and physical characteristics of the track. All of these factors must be considered when setting up a rail grinding program.

MEMOIR

Lauress Charles Collister

1915-1984

Lauress Charles Collister, retired manager of the Tie and Timber Department for the Atchison, Topeka & Santa Fe Railway Company, died Thursday, August 2, 1984.

Lauress was born on September 12, 1915 at Paonia, Colorado.

Lauress started work for the Atchison, Topeka & Santa Fe Railway Company as a chemist in the Tie and Timber Department in 1940, after graduation from Iowa State University where he received a degree in forestry. In 1942 he joined the U.S. Navy and was discharged in 1945 as a Lieutenant. After his return to the Santa Fe he served in various capacities until he was promoted to the manager of treating plants system in 1955 and manager of the Tie and Timber Department in 1972. After his retirement in 1977 he was a consultant to the Association of American Railroads.

Lauress became a member of AREA in 1948 and Committee 3 in 1950. He was also a member of the old Committee 17, Wood Preservation until it was combined with Committee 3. He was a past president of the American Wood Preservers Association and recipient of their award of merit in 1983. He was a very active member of the Railway Tie Association and was a recipient of their "Branding Hammer" award. Lauress married Enla Murray in 1945 and they have two sons.

Lauress is and will be missed very much by his many friends and associates in the AREA, railroad industry and the wood treating industry.

J.B. Miller

DIRECTORY OF CONSULTING ENGINEERS

NOTES

American Railway Engineering Association

Bulletin 698

DECEMBER 1984

Proceedings Volume 85 (1984)

CONTENTS

Cover Photo: Ferrocarriles Nacionales de Mexico employees with special A.R.E.A. train from Mexico City to Queretaro wait at Tula Station while delegation visits bridge projects nearby, October 26, 1984.

Back Cover: High bridge under construction near town of Tula on high-speed Mexico City-Queretero Line (note S-curve in background and archaeological ruins on hill above and to right of second pillar).

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Presentations made at the 1984 Regional Meeting which do not appear in this *Bulletin* will be published in a later issue.

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DEDICATION

This A.R.E.A. Bulletin is Dedicated to

EDUARDO A. COTA

Director General

Ferrocarriles Nacionales de Mexico

for his untiring devotion to his railroad and his country
and his support of the A.R.E.A. and its 1984 Regional Meeting
October 25-27 in Mexico City

Scenes From The 1984 Regional Meeting October 25-27, Mexico City, Mexico

A.R.E.A. President Terrill and NdeM Director General Cota toast the friendship between the railroaders of Canada, Mexico and the U.S.A. during luncheon between technical sessions in Mexico City, October 25.

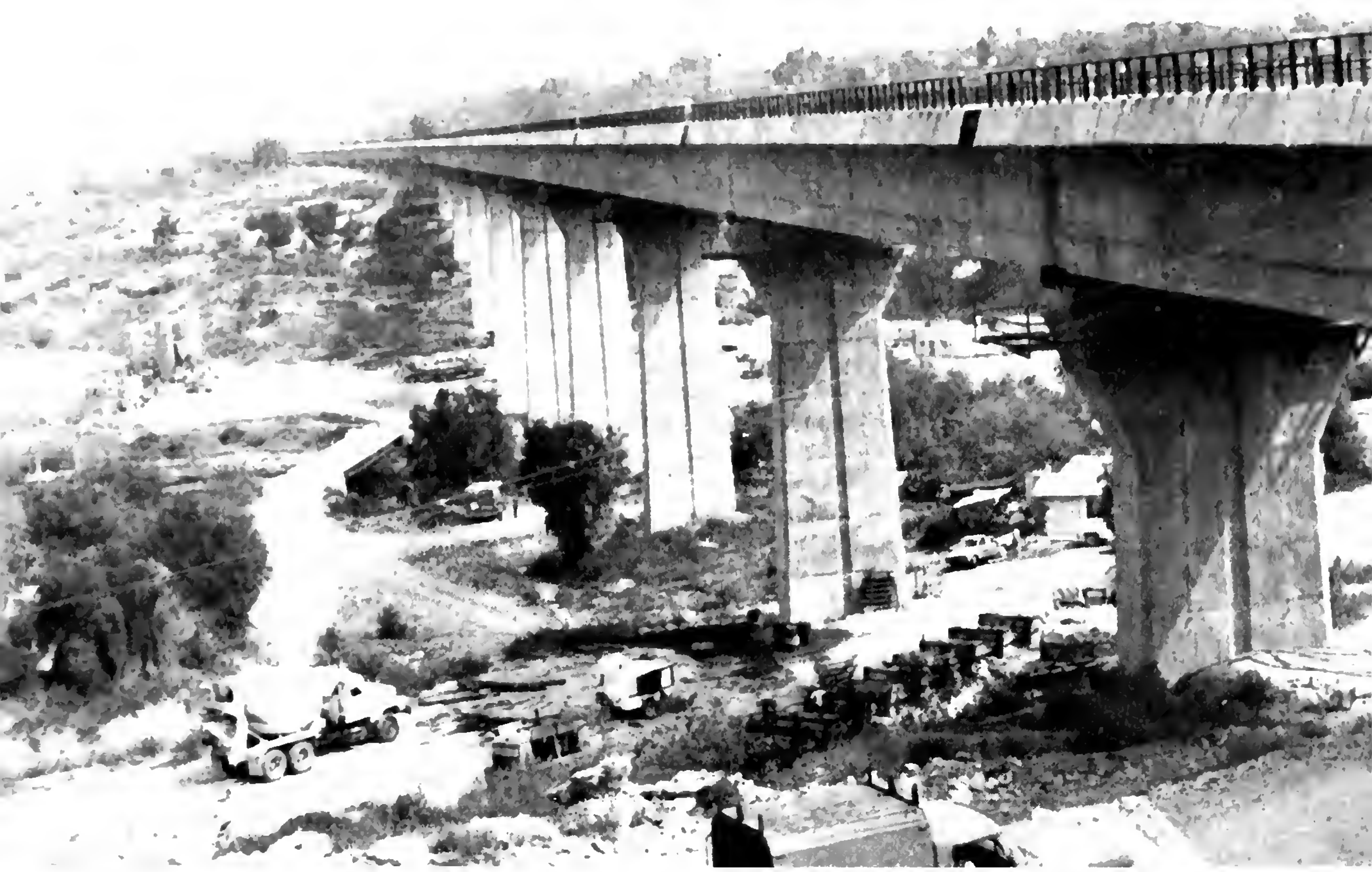
Sr. Ruiz Castro of NdeM addresses technical session.

Phil Richards of CN explains Canadian railroading to technical session.

On the second day of the meeting, October 26, there was a train trip from Mexico City to Queretaro to view construction of the new high-speed line and to ride over parts of it.

NdeM Director General Cota and A.R.E.A. President Terrill at Buenavista Station prior to departure of special train for Queretaro.

Group leaving train at Tula Station to view bridge projects in this vicinity.



High bridge at Tula on high-speed Mexico City-Queretaro Line (note S - curve in background and archaeological ruins above and to right of second pillar).

Cut just west of Tula high bridge with track under construction.

A.R.E.A. special train during stop at San Juan Del Rio.

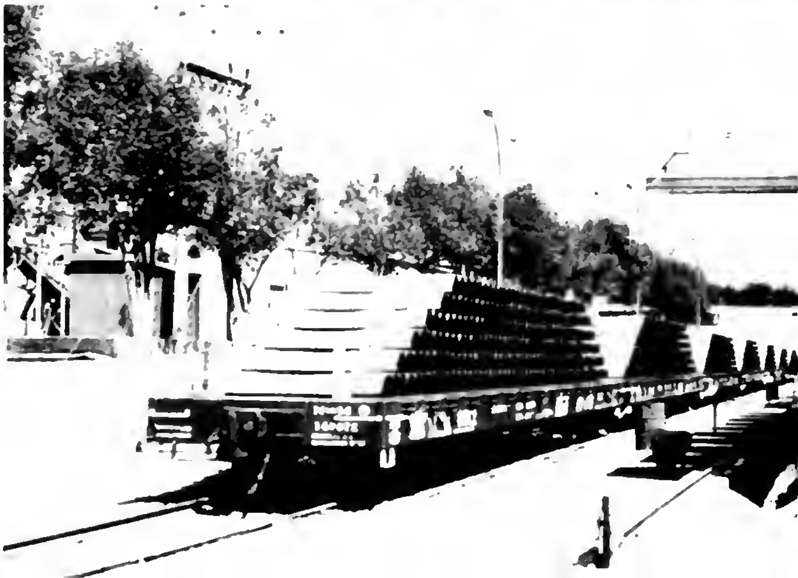
Old and new line as seen from rear of special train (two block ties, though no longer being purchased, were used in earlier construction on new line).

Banquet was hosted by NdeM in evening at the Hacienda de Los Morales Restaurant in Mexico City. Seen here is the head of the Chihuahua Pacific Railroad, Sr. Salmeron and his wife who were guests of honor.

Visit to I.T.I.S.A. Concrete Tie Factory

Members of group within factory complex.

I.T.I.S.A. employee walks on concrete ties minutes after demolding, which took place within minutes of concrete heing poured into mold.



NdeM flatcars loaded with new concrete crossties ready to leave plant.

Presidential Presentation

V. R. Terrill*

It is certainly a pleasure to be here at the first A.R.E.A. regional meeting in Mexico. As the president of the American Railway Engineering Association, I want to publicly give Mr. Eduardo Cota, Director General of the National Railways of Mexico, deep thanks for all the efforts that he and others from the National Railways of Mexico have made in arranging for this conference, and especially for the train trip over a portion of his system tomorrow. I have happily anticipated that trip since I viewed the spectacular slides shown in Chicago two years ago. Mr. Cota's dedication to this railroad and his long working hours are famous all over the continent and are an inspiration to railroaders everywhere. Perhaps we might convince him to slow down his pace and spend more time in hosting his U.S. and Canadian railroad allies on rail trips through beautiful Mexico.

We also wish to thank Mr. Dondish, President of Industrial Enterprises of the Mexican State of Tlaxcala, for the fine arrangements he and his people have made for Saturday. Our thanks also to Knox Kershaw, President of the Railway Engineering-Maintenance Suppliers Association, for the coffee bar which they sponsored during the technical conference today. Lou Cerny, our Executive Director, was forced on two occasions to visit Mr. Cota and enjoy this beautiful historic city while coordinating this historic meeting. We thank him and his staff for their efficient contribution.

Mexico is a full partner in the railway network of our great continent. Railway equipment can and does move freely without modification among the railways of the three great nations of our continent. We know that this meeting will strengthen this continental partnership.

The AREA which stands for American Railway Engineering Association is an organization of approximately 4200 men and women who are engaged in railway civil engineering and associated fields, primarily subgrade, track structure, and bridges and the equipment and materials necessary for maintenance on the 1435 millimeter or 56.5" gauge railway network of Canada, Mexico, and the United States. The objective of the AREA established at its founding in 1899, 85 years ago, is the advancement of knowledge pertaining to the scientific and economic location, construction, and maintenance of railways.

AREA's objective is achieved through the work of a Board of Direction, a headquarters staff, and 22 technical committees. The storehouses for the accumulated knowledge are found in the *Manual for Railway Engineering*, which has been published since 1905, the *Portfolio of Trackwork Plans*, and an annual series of *Bulletins*. The technical committees, which are limited to a maximum of 100 members each, operate through about 160 subcommittees. The committees, except for new Committee 2 and Committees 24 and 32, are each responsible for a chapter in the *Manual*, which has a corresponding number to that of the committee. Committee 5, in addition to *Manual* Chapter 5, is responsible for the *Portfolio of Trackwork Plans*.

Committee 1 deals with ballast and the subgrade on which it rests, as well as culverts, embankments, and rock cuts.

Committee 2, which was established this year, handles matters involved with track measurement devices, especially track geometry cars.

Committee 3 is concerned with timber crossties and wood preservation.

Committee 4 is in charge of rail, including cross-sectional design and metallurgy, and

Committee 5 deals with the overall track structure.

Three separate bridge committees exist, numbered 7, 8 and 15. Committee 7 is concerned with timber structures such as the trestle on which a welded rail train is setting rail. Committee 8 is concerned with concrete structures including retaining walls, and all types of foundations, whether

*President, AREA; Vice President-Engineering, Boston & Maine Corp., and Chief Engineer, Maine Central concrete or not, and Committee 15 handles steel structures, as well as determination of bridge impact loads.

Committee 10 handles concrete tie matters, including fastenings.

Committee 14 deals with yards and terminals.

Committee 33 deals with electrification.

Committee 6 deals with specialized railroad buildings.

Committee 27 deals with specialized railroad maintenance equipment,

and

Committee 9 deals with highway-railway crossing matters.

Items handled by still other committees include Committee 11, Engineering Records & Property Accounting; Committee 13, Environmental Engineering; Committee 22, Economics of Plant, Equipment & Operations; Committee 16, Economics of Railway Construction & Maintenance; Committee 24, Engineering Education; Committee 28, Clearances; Committee 32, Systems Engineering; and Committee 34, Scales.

As I mentioned before, the A.R.E.A. issues several publications, including the *Manual for Railway Engineering,* which contains specifications, rules, plans and instructions developed by the technical committees and adopted by the Association as "Recommended Practice."

We also issue a *Portfolio of Trackwork Plans,* which is used as a reference in railway construction and is a companion volume to the *Manual.*

The A.R.E.A. has also published the *Talbot Reports.* The *Talbot Reports* are the findings of a special committee which functioned from 1918 to 1942 investigating stresses on railroad track. This investigation was one of the most extensive searches into track structure behavior ever undertaken. The Committee's primary function was to test U.S. track in an attempt to characterize track response to railway loadings. While 25 years seems like a long time for such a study, in this case, it ended much too soon. It perhaps should have adjourned for 20 years and started again in 1962 to study the ever increasing loads and the differing vehicles carrying those loads. This in turn might have allowed more U.S. maintenance people to grow old gracefully instead of aging quickly from derailment to derailment. I would much rather have learned rock and roll by watching my children dance during the 60s and 70s than from the harmonious actions of 100T high-center-of-gravity cars dancing over and around 39′ jointed rail.

Nonetheless, the *Talbot Reports* served as a foundation for later studies on track harmonics and were originally published in A.R.E.A. *Bulletins.*

Such technical *Bulletins* are published 4 times a year and include the results of analytical studies, field tests, investigations, and the reports of research studies.

A.R.E.A's monthly newsletter is published in *Railway Track and Structures* magazine. Each member receives a subscription to the RT&S magazine which has been paid for out of the member's A.R.E.A. annual dues. The cooperation between A.R.E.A. and *Railway Track and Structures* magazine has proven itself to be beneficial for both.

The A.R.E.A. is used by the Association of American Railroads as its prime source of advice on railway engineering matters. In turn the A.R.E.A. committees look to research resources of the Association of American Railroads, such as the Technical Center in Chicago or the famous Transportation Test Center at Pueblo, Colorado, in the United States.

The A.R.E.A. maintains a liaison with the Federal Railroad Administration with regard to railway engineering matters in the United States.

The *Manual for Railway Engineering* is intended to be a set of recommended practices. The committees work constantly to update the contents of the *Manual*, which is revised once a year effective August 1, as is the *Portfolio of Trackwork Plans*. A general requirement is that recommendations should be based on proven field experience. Therefore, new products may appear that are more suitable than are the *Manual* recommendations, and the *Manual* is not intended to be used as a barrier to the use of new products. Committee decisions on changes come from the experience of the individual members, their knowledge of recent research and development results, and information they have gathered from many sources.

We are proud of the A.R.E.A. *Manual*, the *Portfolio of Trackwork Plans* and the A.R.E.A. *Bulletins* which are used all over the world as a maintenance-of-way bible and we are especially proud of you, our membership who give so freely of your time to keep the studies, specs, plans and designs updated through the A.R.E.A. committees.

The goal of advancement of knowledge is pursued through technical conferences such as this one and publication of technical papers. The A.R.E.A. *Bulletin*, first published in 1900, is the vehicle for dissemination to the entire membership of proposed changes to the *Manual* and pertinent technical papers. The technical papers include those presented at annual meetings and regional meetings. The annual technical conference is held in Chicago, the home of many major U.S. railroad organizations and the geographic center of the 3 great nations making up the A.R.E.A. Regional conferences, such as this one, are held in a different location each year to provide at least an occasional opportunity for members in different sections of our continent to participate in a technical meeting.

Committees often undertake activities other than work on the *Manual;* for example, the Buildings Committee sponsors a Student Architectural Design Competition. The prize is awarded in March at the annual meeting. The Engineering Education Committee also works with students and university faculties to encourage interest in railroad engineering education and have sponsored various student engineering competitions in the past.

The A.R.E.A. Board of Direction consists of a president, two past presidents, Hubert Rose, Southern Railway, and Mike Rougas, Bessemer & Lake Erie; two vice presidents, Phil Richards, CN, and Herb Webb, Santa Fe; and 13 directors elected by the membership, and a treasurer. I am president for 1984 and 1985 through the March annual conference. On the last day of that conference, the new president will begin his one year term. The director from Mexico is Rafael Tebar Mijares.

The headquarters staff consists of an executive director, Mr. Cerny, a director of engineering, Mr. Grotz, and a manager-headquarters, Ms. Meyerhoeffer, who manages the remainder of the staff. Mr. Cerny reports to the Board of Direction.

In 1979, the A.R.E.A. headquarters staff offices were moved from Chicago to Washington, D.C. This move, as expected, has improved coordination between A.R.E.A. and A.A.R. and the Federal Railroad Administration of the United States Government and through the Government to the railway agencies of Canada and Mexico.

A.R.E.A. is looking forward to a future of continued service to the railway engineering profession, to the railroad industry, and to society as a whole, as it innovates and promotes progress on the 1435 millimeter gauge railway network of the three great nations of our continent.

On behalf of the A.R.E.A. Board of Direction I wish to thank Eduardo Cota, Director General of the National Railways of Mexico, for the work that he has done in making it possible to hold the 1984 A.R.E.A. Regional Meeting in this beautiful city and in this great nation.

We eagerly look forward to the Canadian presentation and beyond to learn more about Mexican railways and maintenance of way practices from the papers and presentations of the Mexican engineering professionals to follow. Thank you.

Headquarters' Report

L. T. Cerny*

Nosotros, los Ferrocarrileros de Canadá, México y los Estados Unidos tenemos una red ferroviaria en tres países. Desde el calor estival de las llanuras costeras de México hasta el frío invernal de las latitudes septentrionales, somos una red. Desde los vastos horizontes y el silencio de nuestros desiertos hasta el clamor y emoción de nuestras grandes ciudades, somos una red. Nosotros, los Ferrocarrileros de Canadá, México y los Estados Unidos nos damos mutuamenta la bienvenida al iniciar esta reunión.

Deseo expresar mi profundo reconócimiento al Sr. Cota por su aliento y apoyo a esta reunión, y a todas esas otras personas en los Ferrocarriles Nacionales de México que tanto han contribuido a hacer de esta reunión un éxito, especialmente el viaje de mañana a Querétaro.

Es muy importante que los ingenieros mexicanos formen parte de la corriente principal de desarrollo de la ingeniería ferroviaria en este continente, y el personal de la sede de la AREA se esforzará para hacer que no se interpongan al logro de este objetivo situaciones tales como diferencias de idioma, retrasos en el correo y aspectos financieros. Esperamos que siga aumentando el número de nuestros miembros mexicanos, así como el de los miembros mexicanos que formen parte de nuestros comités técnicos. Estoy seguro de que su Jefe del Departmento de Vía y Estructuras, Sr. Tebar, que es nuestro nuevo miembro mexicano en el Consejo Directivo de la AREA, podría ayudar en este aspecto.

Los Ferrocarrileros mexicanos tienen mucho que enseñar a los Ferrocarrileros de los Estados Unidos y de Canadá, y viceversa, y es mediante nuestra asociación contínua en la AREA como podemos ayudarnos mutuamente. El personal de la sede de la AREA tiene por finalidad ayudar a los miembros de la AREA a alcanzar estas metas; por consiguiente, sientanse con la libertad de recurrir a nosotros si podemos ayudarles.

Canadian Railroad Perspective

P.R. Richards**

Mr. Chairman; Engineer Rodolfo Felix Valdes; Sr. Eduardo A. Cota; Mr. Vincent Terrill; Ladies and Gentlemen.

It is a great pleasure for me to be here in Mexico for the 1984 A.R.E.A. meeting.

In order to further solidify the friendship between our countries, several Canadian railway officers have visited your country during the past few years. We have always found that a better rapport has been established when our presentations have been made in your beautiful language. For this reason, I am introducing my colleague, Mr. Hermann Paffrath, who is known to many of your railway officers, whom I have requested to make the presentation in Spanish on my behalf.

It is my honour today to talk about the Canadian perspective of railroading. This presentation covers, in a very general way, an overview of our operations as well as some engineering techniques and projects that have been designed towards achieving maximum productivity.

With a population of 24 million people, Canada is one of the largest countries in the world. Spanning 5,400 kilometers from Atlantic to Pacific Oceans, its vast network of railroads also extend northward into "perma-frost" territories that are frozen 8 months of the year and encounter some very high temperatures to the south.

*Executive Director, AREA, and Executive Director, Engineering Division, AAR

**Chief Engineer, CN Rail

The trackage goes along coastal areas through mountainous terrain, over prairies and farm country and serves metropolitan areas such as Montreal, Toronto, Vancouver, and Halifax, as well as small communities and some very isolated areas.

Canada is an exporting country with a large mixed manufacturing industry, mainly in the urban areas. We have large resources of grain, minerals and fuels, most of which is carried by rail. There are two major railroads in Canada.

CP Rail, formed in 1881, became Canada's first transportation link between east and west and now comprises 24,500 kilometers of main line trackage.

CN Rail, owned by the government of Canada, was incorporated in 1922 by joining a number of smaller railroads into a very large system of 37,800 kilometers of main track and forms the second transcontinental system.

Although smaller in size in comparison to CP and CN Rail, other important railroads such as B.C. Rail has 2,700 kilometers servicing the central and northern regions of British Columbia.

In the east, Ontario Northland, QNS&L, and Quebec Cartier also provide vital rail linkage into the northern areas of their provinces.

Along with another 2,500 kilometers of short line railroads, Canada has 71,000 kilometers of main route trackage over which its railway companies handle the important business of moving freight. Passenger traffic is handled by another company called VIA.

Stretching from one end of the country to the other, 77,000 employees strong are involved in hauling Canada's wealth from producer to marketplace. Today, railroading is a fast moving, complex business that demands "state of the art" technology and constant innovation.

Everyday, some 500 trains run over our main lines. We have a fleet of 3,200 locomotives and 165,000 freight cars which include box cars, flat cars, tank cars, coal and ore carrying gondola cars and other specialized equipment. To manage this fleet we use sophisticated computer systems which centrally monitor the movement of trains 24 hours a day, 7 days a week.

This system incorporates automatic signalling which is used in corridors with a high volume of traffic. The train dispatcher sits at the control panel where trains are displayed in schematic form and monitors their movements by the position of the lights on the panel.

The railways use associated computer systems to marshall and dispatch trains, maintain car inventories and schedule maintenance and repairs. These systems are present at our yards, shops, and regional and system offices.

To enhance efficiency and safety, employees must be well trained in uniform code of operating rules which is the standard reference for train and yard operations of all Canadian railways.

Another important tool is our radio equipment which provided all freight trains with end-to-end and switching communications. All major yard crews and maintenance of way supervisory personnel are also equipped with this valuable communications device.

Our long-term commitment to equipment renovation and plant expansion make the railways a major purchaser of Canadian goods and a significant contributor to the economy. World demand is growing for our natural resources, grain, coal, sulphur, potash, forest products and petro chemicals.

In western Canada, in the late 1960s there was a boom in demand for bulk commodity movement, like coal and potash. For this reason the railways introduced unit train operations. This new concept in rail transportation has improved productivity. And like a conveyor belt a single commodity on a dedicated train carries a load from source to destination, over and over again.

To carry longer, heavier trains the railways have increased the amount of ballast under the ties on mainline routes. The standard of 30 cm has doubled in the last few years. Concrete ties are laid on curves of 2 degrees or more carrying an annual tonnage of 20 MGTMs or more.

High mechanization brings higher productivity. With the use of automated systems such as the P-811, worn rail is replaced with new continuously welded rail on concrete or wooden ties. Several miles of track can be changed in a single 8 hour work period, enabling the railways to get on with the job of moving freight.

The concept of the P-811 has been modified to create a rail change out (RCO) machine for use in those areas where concrete ties are not specified. In this case, the equipment for removing the wooden ties and placing concrete ties is replaced with equipment to prepare the wood ties to take new rail.

The mechanization of track renewal or maintenance is further enhanced by the use of the track recorder car. This car records track alignment, super-elevation, gauge, surface of each rail and cross level.

It contains a computer which analyzes all data and produces track-defect reports. This data enables us to monitor track conditions so as to avoid possible damage to property.

Major Canadian projects include a $600 million investment made by CP Rail on Rogers Pass. The project involves the building of a reduced grade westbound track for 33.5 km. The existing single track line which goes through the Connaught tunnel will be used for eastbound trains. One tunnel on the new line will be 14.58 km long, making it the longest railway tunnel in North America.

The main reason for the building project is a capacity limit at Rogers Pass with its steep grade ranging from 2.0% to 2.44%. Currently all westbound freight trains must be pushed by an additional six 3,000-horsepower diesel locomotives. The returning movements of these pusher units cause delays on the single track mainline.

The major tunnel will feature a unique ventilation system that will allow twice the train frequency.

Another major construction project has been undertaken by B.C. Rail called the Tumbler Ridge branch line at a cost of $450 million.

The northeastern region of British Columbia contains large quantities of high quality metallurgical coal. In order to market this commodity, it was necessary to provide rail access. B.C. Rail undertook to build a new line originating at Anzac which included eleven bridges, three tunnels, new construction of grade, track structure and access roads.

A study determined that for this particular project, electrification was more economically feasible than diesel fuel as the cost of fuel is a major consideration in railway operations. This also eliminated the need for costly tunnel ventilation systems which would be required for diesel engines. Power stations were constructed to accommodate these electric units.

Once B.C. Rail loaded the commodity, it is routed in Prince George, where CN Rail continues its movement to Prince Rupert for shipment to large export markets such as Japan.

To accommodate the movement of coal and other commodities to the west, it is necessary to upgrade the facilities in northern British Columbia. This is CN Rail's portion of the huge megaproject to handle coal and grain to Prince Rupert, in a section of the country that is experiencing a growth.

With more and more demands on capacity because of these growing markets, CN launched a systematic program to expand track capacity in the west. The plan involves selective double tracking and terminal expansion, carefully gauged to the most up-to-date traffic. By the end of 1983, CN has double tracked 309 km. and has expanded 19 terminals from Winnipeg to Vancouver. In this decade, CN Rail will be spending almost $1.4 billion to accomplish the task.

In summary, I would like to further touch on matters related to Canadian railway engineering. Each of the two major railroads spends about $250 million annually on plant replacement materials. Most of this is for new rail, ballast, ties, bridges, signal control systems and communications. In addition to this, they each also spend about $500 million on maintenance of these fixed plant systems.

In curves, on our high density lines carrying 20 MGTM to 65 MGTM, it is standard to use premium quality rail such as chrome alloys, head hardened and in the future, fully heat treated. We are using CWR (continuous welded rail) on all our relays (new and part).

The rail expansion/contraction problem is very critical in Canada with temperatures varying from −40 degrees celsius to +60 degrees celsius. This differential of 100 degrees obliges careful inspection to prevent track buckles in hot weather and pull aparts in winter time.

We use mainly hardwood ties in the eastern half and softwood ties in the west because of availability. Each railroad replaces about 2 million wood ties annually.

CN Rail has about 2.5 million concrete ties, mainly on curves in territories carrying 20 MGTMs and higher and has a concrete tie/rail installing machine for about 300,000 concrete ties annually on high density heavy curvature lines.

We use high production ballast undercutter cleaning equipment plus high speed ballast tampers.

With the use of heavy 100-ton cars on long unit trains, we utilize the strongest, longest lasting track materials to lengthen the replacement cycles to reduce on-track time, thereby resulting in minimum interference to high-density train traffic. High quality preventive maintenance is very important to us to make the plant last as long as possible in order to defer replacement costs and gang time on track.

Bridge replacement work is very extensive on both CN and CP rail, varying from 30 to 60 units annually on each railroad.

CN Rail is building two new geometry cars utilizing modified 100-ton cars which have been instrumented to give us the true 33 ton per axle load defect readings.

I hope this gives you a better view and understanding of Canadian railroads. Again, thank you greatly for the opportunity to be here with you today.

Overview of the National Railroad System in Regard to the General Plan for Its Modernization and Particularly the Renovation of the Mexico-Coatzacoalcos Route

Ing. R. Ruiz C.*

For more than a decade, the Mexican Government has been trying to give the railroads a new image marking the beginning of their modernization, and updating them in accordance with traffic needs and in agreement with the policy of efficiency and development of freight transportation.

For many years few investments in the railroad plant were made, which created a serious hold back. But in view of the present land transportation problem, more resources are being assigned to the development of the following programs:

— Construction of new double track lines (in high density traffic areas), including their electrification of approximately 850 KMS (530.3 miles).

— Localization and construction of approximately 350 KMS (220.7 miles) of new lines.

— Rectification works to better grades and curvatures in 275 KMS (172.2 miles).

— New yards and large classification terminals.

*Assistant Director, Track and Structures, National Railways of Mexico

Meanwhile, within the institutional framework, the renovation of several lines of the system have been scheduled. One of them corresponds to the so-called "Southeastern Door" between Cordoba, Ver., and Coatzacoalcos, Ver., through which petroleum and chlorine unit trains run, through fast freighters, passengers, trains to both the southern border and the Yucatan Penninsula, and short-distance trains of the sugar cane area. The important traffic is generated by the petrochemical industry, by the industrial complexes Pajaritos, Cangrejera and Morelos in Coatzacoalcos, Ver., as well as by the industrial corridor that exists between this city and Medias Aguas.

— Along that route of traffic 210,000,000 gross ton-KMS/month is being moved, which is equivalent to approximately 7.0 million GT/annum. It has a very promising future, in view of the development poles which that part of the national territory will have.

In view of the importance of these lines and independently of the existing projects for new localizations with better geometrical design, it has been decided that there is urgent need to carry out in the shortest possible time the rehabilitation of the track as well as to reinforce and augment the load capacity of the bridges. Since traffic density is increasing and at present neither the track nor the bridges are in condition to stand such traffic without operational problems, we are hoping to resolve the serious aspect of costly accidents that are occurring in that area with the utilization of machinery that will allow major advances.

The economic development of the country during the last part of the past century and the first part of the present century fundamentally rested on the construction of the railroads whose progress was accelerated.

The Mexican railroad network at present consists of 4 companies:

Ferrocarriles Nacionales De Mexico

Ferrocarril Del Pacifico

Ferrocarril Chihuahua Al Pacifico

Ferrocarril Sonora-Baja California.

Ferrocarriles Nacionales De Mexico at present has 15,480 KMS (9,620.9 miles) of main line and 3,525 KMS (2,190.8 miles) of secondary tracks. The other railroads have 4,426 KMS (2,750.8 miles) of main line and 937 KMS (582.4 miles) of secondary tracks, including 10,020 bridges and 17,000 culverts of which 38% are provisional wooden structures, together forming the great longitudinal and transversal railroad axis that cross our country. The system is used for the massive transportation of people and merchandise at a low cost, connecting the capital of the republic with the majority of the states' capitals, the border and maritime ports, as well as the main cities of the country and numerous communities throughout approximately 750 stations.

Ferrocarriles Nacionales De Mexico's present administration has implemented a very ambitious program with regard to the rehabilitation and maintenance of tracks. This program is mainly being implemented in the southern part of our country with the end result being the upgrading of them to the reality and needs of present and future traffic. At the same time, we are trying to resolve the accident situation that exists due to the physical conditions of the facilities and are trying to increase the transportation capacity and efficiency in that region. As a consequence new fronts of work will be open to lay new rail in lines. We are using second hand rail to rehabilitate some lines. Complementing all this is an intensive program of rehabilitation and construction of bridges and culverts to replace those of low capacity and with many years of service in lines that operatively restrict the movement of longer and heavier trains.

The route Mexico to Coatzacoalcos is integrated by several sections with different geometrical characteristics.

One section crosses the high Mexican plateau between Mexico and Esperanza and descends through a stretch of mountainous territory which is difficult for railroad movement. It proceeds to the coast between Esperanza-Puebla and Cordoba, Ver., and has grades in the area of 4.1% and curvature of 12 metric degrees (18.2° English grades).

The remaining line goes from Cordoba, Ver., to Medias Aguas, Ver., and Medias Aguas Coatzacoalcos, Ver. It has grades and curvature degrees lower than the aforementioned lines with only some places of difficulty for the operation due to the hilly country they cross.

The railroad general division under the Communications and Transportation Secretary has under construction a new line close to Los Reyes, Pue. (near Esperanza, Pue.) and Cordoba, Ver., with a compensated grade of 2.5% and maximum curvature of 6 degrees, which will allow us to diminish the difficulties of the operation in this mountainous territory. In the near future we will initiate the construction of a new line between Teothihuacan and Los Reyes with a maximum grade of 1.0%. We will make the total rehabilitation of the remaining sketches aforementioned specifically consisting of the integral renovation of the track. Welded rail will be used with new 115 lbs/yd. rail, concrete ties and elastic fastenings.

Traditionally since the early beginnings, the maintenance and rehabilitation works were made by labor, using a great number of track workers to make the various phases of these works resulting in low productivity and quality not precisely optimum.

To increase the productivity of the track it is necessary to integrate section gangs and distribute them in the divisions of the system. Half a century ago the gangs used to work with rail of low weight (40 to 70 lbs/yd.) and with a traffic much lower than at present, and it is worth pointing out that they were able to maintain the track in acceptable conditions up until the 50s.

Later the transportation needs in our country made it necessary to rehabilitate on a great scale and to lay higher caliber rail up to 100, 112 and 115 lbs/yd. This higher caliber rail is presently installed in almost 90% the system. This required that we augment the workers' productivity and the need to integrate section gangs. These gangs are composed of groups of 20 to 25 men responsible for 40 to 50 KM. We provide them with enough light equipment such as rail cutters, portable tamping machines, rail drill, tie drill, etc. They are backed up by an integral group of heavy machinery to do the leveling and alignment work which is required by the heaviest traffic lines and those with deference in their maintenance.

To comply with this commitment and carry out the established programs, a total mechanization of the track rehabilitation works is required, which under normal conditions would allow us to change 2 KM of rail per work shift.

In modern tracks in which heavier elements are used, long welded rail and concrete ties which are difficult to handle by hand, the need has arisen to utilize machinery more varied and specific. The main objective is not to extenuate the personnel that directly perform this heavy work while at the same time increase quality and productivity.

In work on this kind of modern track, the equipment varies specifically in the first 2 phases. Existing renovation trains like the "SECMAFER" consists of the following equipment:

— a positioner,

— two gantry cranes connected with a beam,

— ballast crader and compactor, and

— a rail layer.

We recognize that it is very difficult to explain in a few words the scope of activities and means of realization that Ferrocarriles Nacionales De Mexico has. It has been our desire to present to you a brief description of some of the main works under construction and to emphasize the importance that the mechanization of the track rehabilitation work has had in bringing about the advances that are required by the railroad in our country.

Thank You.

History of the Application of Concrete Ties in the Mexican Railway System

Ing. R. Tebar M.*

This work hopes to give you an idea of the development that the Méxican railway system has made with the use of concrete ties and its fastenings since it started a few years ago.

Taking into consideration the advantages of continuous welded rail and elastic fastening over concrete ties (usually called "modern track or elastic track"), the Mexican National Railway System over 20 years ago started the use of this type of track, considering as a main point, constructing that type of track over concrete ties starting at San Carlos, Coah., up to Ciudad Acuña, Coah., a bordering city. Once this line was finished in 1960, it was given to our company for operation. The line is 39 Km. (24 miles) long, and is constructed with a maximum curvature of 2°. The ties used are Bi-Block.

There have been several factors that have influenced our railway system on the construction of this type of track, using concrete tie and elastic fastening, relieving in this way the use of "classic-track" constructed with wood tie and track nail. Between these factors we can quote the differed protection given to the relief of the wood tie in bad conditions. This problem has been caused because of various circumstances, such as forestal problems, public land problems, delayed delivery time from the suppliers, lack of economic resources, not being able to ask for the quantities to fulfill our needs to relieve this type of tie in our track rehabilitation program as well as on our track maintenance programs.

A very important factor that influences and also has an advantage on the use of concrete ties is their durability or its "useful life period". In theory, the concrete tie has an average life of 50 years while the wood tie is considered to have an average life of only 15 years.

A method used in laying concrete ties is done with the well-known "SECMAFER" equipment, created specially for the costruction and rehabilitation of track with continuous welded rail over concrete ties. In 1969, the Mexican National Railway System acquired two of these units with which an average of 1,100 mt. (1,202 yds.) in a 6-hour work period can normally be obtained. We can mention a second method organized by the Mexican National Railway System and which has given satisfactory results. It consists of the use of the following group of medium machinery:

1 - Burro Crane,

2 - Scarifier - Inserters,

4 - Spike Pullers,

4 - Tie Inserters, and

6 - Power Track Wrenches.

All of these machines are operated by trained track personnel.

With the SECMAFER the concrete ties are transported by means of special gantry directly from the platforms where they have been conveniently stowed to where they are unloaded on the field. The tie having been previously unloaded from the platforms by means of the Burro Crane is layed on the side of the enbankment with adequate spacing and line up so that afterwards they can go ahead with the general jobs with the tie inserter machines watching over the spacing and line up of the ties to receive the continuous welded rail.

There are 1,522,362 ties of the Bi-Block Type, and 3,965,960 ties of the DYWIDAG Type on the system.

This year we expect to continue to lay 30,000 to 40,000 of this type of tie on the Gulf, and SE-VCI division in the rehabilitation of the track with continuous welded rail.

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*Chief Engineer, Track and Structures, National Railways of Mexco

We have been using ties type Bi-Block "RS," "SL" and S-75 and type Monoblock "DW." The following are some characteristics of each type.

"RS" and "SL" Bi-Block Type Tie

Weight	210 Kg. (462.91 lbs.)
Compression Strength	490 Kg./cm.^{2} (6970 lbs./in.^{2})
Central Rod (tensile limit)	2,520 Kg./cm.^{2} (35,842)
Height (under rail)	205 mm. (8.0709 inches)
Upper Width (under rail)	220 mm. (8.6614 inches)
Lower Width (under rail)	290 mm. (11.4173 inches)

"DW" Monoblock Type Tie

Weight	247 Kg. (544.53 lbs.)
Compression Strength (28 days)	600 Kg./cm.^{2} (8530 lbs./in.^{2})
Tensile Strength (7 days)	65 Kg./cm.^{2} (925 lbs./in.^{2})
Height (under rail)	205 m. (8.0709 inches)
Upper Width (under rail)	210.0 m. (8.2677 inches)
Lower Width (under rail)	317.5 m. (12.5 inches)

This last tie is the one that has been used in a greater scale on our system. The length has had small charges to obtain a strengthened tie and with more flexion resistance.

"DW" Monoblock Type Tie (Strengthened)

Weight	275 Kg. (606.26 lbs.)
Compression Strength	600 Kg./cm.^{2} (8530 lbs./cm.^{2})
Tensile Strength	65 Kg./cm.^{2} (925) lbs./cm.^{2})
Reinforcement Steel Rod	10.0 m. 0
Height (under rail)	205 m. (8.0709 inches)
Upper Width (under rail)	210 m. (8.2677 inches)
Lower Width (under rail)	317.5 mm. (12.5 inches)

In general, we can assure that the use of the Monoblock "DYWIDAG" tie has given very good results in its structural properties. In some cases it is installed in sections where there are barely 10 cm. of ballast, in zones where there is drainage deficiency, or where there is an obstruction such as a ditch causing failures to the track which causes adverse situations of track maintenance.

Even though this tie exhibits fatigue caused by tension on the concrete at the point of the rail base with fissures that go all the way to the upper part of the tie, the Mexican National Railways has designed a strengthened tie to control this problem.

Regarding the Bi-Block ties, it has been observed that they are seriously damaged when they have been installed under defects of the rolling surface of the rail. An example of this is a bad welding. They are affected and destroyed because of flexure under the rail line of the steel rod with the blocks or accelerated deterioration and the lower fare by the above effect until the reinforcement steel rods are exposed because of the low quality of the concrete. These defects are to be eliminated reinforcing the details we first mentioned, such as increasing the line width, increases the diameter of the steel rod, etc.

In relation to the fastening accessories that consists of a bolt, elastic clamp "RN", seat and the rubber pad, in general we have seen good results having very little maintenance costs on this accessory. On the other hand, the "RN" fastening for its application on the Monoblock Type Tie "DW" has been improved, being of the weight of the tie by adapting a record blade, and achieving a longer life period of the fastening.

In 1985, there will be a program to continue the rehabilitation of lines on the SE-NT Division, with continuous welded rail and approximately 300,000 concrete ties type "DW." It's precisely in this zone, where we hope to overcome problems such as insufficient wood ties, and the destruction of the wood ties caused mainly by the tropical weather with long heavy rain seasons.

It is easy to think of the improvements that we have obtained with modern track were the joint-bars have been eliminated. If joint bars have not been eliminated the equipment that runs over these tracks causes a constant impact creating damage as the traffic increases. This in turn increases maintenance jobs. With the impact of the traffic of trains, a joint with joint-bar over a wood tie under certain conditions can accelerate the damage to the track structure. At this point and may cause shearing of the tie, loosening of the joint-bar bolt, deterioration of the ballast base and rail creepage sometimes causing the loss of the superficial part on the rail head, increasing the possibilities of a crack in the rail on the joint. The elimination of these risks has been a great achievement obtained with the use of track with continuous welded rail over concrete ties.

We show, as an example, the variation of the commercial load traffic density, expressed in thousands of tons. Where we can see the strong increase that we have been having since 1960 to this date is on "G", before the Tierra Blanca and Tres Valles-Medias Aguas Section, as well as on line "V" of the Jalapa Division on the Jalapa-Veracurz Section which has forced us to establish the use of continuous welded rail over concrete ties.

On the first chart, we see line "G" of the SG-VCI Division. In 1960 we had a value of the load traffic density of 85.7 thousand tons. For 1983 it went up to 619.3 thousand tons which shows an increase of density of 533.6 thousand tons in 23 years.

CHART 1

DENSITY OF COMMERCIAL TRAFFIC IN DIVISION SE-VCI LINE "G" (1960 to 1983)

YEAR	TIERRA BLANCA-TRES VALLE THOUSAND TON LOADS	TRES VALLES MEDIAS AGUAS THOUSAND TON LOADS
1960	85.7	85.7
1961	189.4	103.7
1962	211.4	211.4
1963	252.7	252.7
1964	248.8	236.8
1965	242.7	243.9
1966	239.3	239.3
1967	227.4	227.4
1968	249.4	210.4
1969	391.1	392.1
1970	270.6	270.6
1971	778.0	285.1
1972	1,198.5	310.4
1973	627.6	395.2
1974	880.5	402.2
1976	569.5	481.9
1977	437.5	645.6
1978	458.2	530.6
1979	528.3	513.7
1980	384.5	618.3
1981	504.1	727.7
1982	602.1	713.6
1983	625.3	619.3

(CHART 1—CONT.)

DENSITY OF COMMERCIAL TRAFFIC IN DIVISION JALAPA, LINE "V" BETWEEN JALAPA - VERACRUZ

YEAR	THOUSAND TONS LADAS RUMBO - AL NORTE	THOUSAND TONS LADAS RUMBO - AL SUR	TOTAL TON LOADS
1960	84.2	41.9	126.1
1961	86.9	23.2	119.1
1962	163.8	84.6	248.4
1963	208.2	120.4	328.6
1964	199.5	129.2	328.7
1965	196.7	161.1	357.8
1966	184.2	152.1	336.3
1967	194.3	169.1	363.4
1968	196.9	161.6	358.5
1969	201.8	151.6	353.4
1970	314.0	190.0	604.0
1971	270.7	187.9	458.6
1972	328.1	231.7	559.8
1973	404.0	255.2	659.2
1974	477.8	302.0	779.8
1975	464.6	281.7	746.3
1976	462.1	236.7	698.8
1977	396.9	239.6	636.5
1978	511.3	344.0	855.3
1979	460.2	381.3	841.3
1980	305.6	342.6	648.2
1981	441.1	356.9	798.0
1982	398.1	330.5	728.6
1983	458.5	281.2	739.7

On the section Tierra Blanca-Tres Valles there was a very high increase in the years 1971-1974, up to a value of 1,198.5 thousand tons in the year 1972. This was probably influenced by the sugar mills that are established in that area.

The line "V", Jalapa Division, the increase is remarkable from 126.1 thousand tons in 1960 to 739.7 thousand tons in 1983, showing an increase of 613.6 in the 23 years.

Chart 2 shows the concrete ties installed on the different lines of the Mexican National Railway System as well as those installed on the Ferrocarril Del Pacifico, Chihuahua Al Pacifico, and Sonora Baja-California, totaling 5,488,322 ties.

CHART 2

CONCRETE TIES INSTALLED ON DIFFERENT LINES OF THE MEXICAN NATIONAL RAILWAYS SYSTEM, FERROCARRIL DEL PACIFICO, CHIHUAHUA AL PACIFICO, AND SONORA BAJA-CALIFORNIA

Division	Line	KMS. With Concrete Ties	Ties "DW"	Ties Bi-Block	Total Ties	Year
CENTRO	"A"	417+435	542,792		542,792	1975/79
GOLFO	"M"	442+978	730,319		730,319	1980/84
GUADALAJARA	"I"	561+492	910,042	22,034	932,076	1972/83
JALAPA	"V"	132+227	165,580		165,580	1974/75
MEXICO	"A"	22+754	37,825			1967/68
MEXICO	"B"	23+400	38,983	—	— 91,802	1967/68
MEXICO	"H"	9+000	14,994			1984
MEXICANO	"S"	59+550	85,008			1970
				—	— 133,322	
MEXICANO	"SH"	29+000	48,314			1977
MONCLOVA	"RA"	39+600		63,360		1960
				—	— 127,644	
MONCLOVA	"RD"	39+366	64,284			1971/72
MONTERREY	"B"	261+520	442,242			1973/74
				—	— 739,206	
MONTERREY	"F"	178+600	208,666	88,298		1971/78
PACIFICO	"NE"	193+831		322,922	322,922	1980
QUERETARO	"A"	32+154	53,568			1970
QUERETARO	"B"	13+000	21,658			1970
				—	— 248,771	
QUERETARO	"AQ"	89+436	149,000			1983/84
QUERETARO	"BQ"	14+733	24,545			1983/84
SAN LUIS	"B"	66+600	139,619			1972
					— 438,642	
SAN LUIS	"BC"	180+000		— 299,023		1972
SE-VCI	"G"	24+500	37,084		37,084	1964
TENOSIQUE	"FA"	10+000	16,660		16,660	1979
FC-PACIFICO		40+470	67,177		67,177	1970/82
FC-CH-PACIF.		437+700		726,725	726,725	1958/82
FC-S. B. C.		101+000	167,600		167,600	1980/81
		3,470+346	3,965,960	1,522,362	5,488,322	

The different locations of these concrete ties on our railway system are the following:

1.- Division Centro-Line "A"

Between the kilometers A-352+800 to A-814+000 in a length of 417+435 track kms. and a total of 542,792 "DYWIDAG" concrete ties. This track was constructed between 1975 to 1979.

2.- Division Golfo Line "M"

Between the kilometers M-9+000 to M-300+000 and from M-457+480 to M-612+040 in a length of 442+978 kms. of track and a total of 730,319 "DYWIDAG" concrete ties. This track was constructed between 1980 to 1984.

3.- Division Guadalajara Line "I"

Between the kilometers I-4 + 000 to I-395 + 125 and from I-439 + 315 to I-594 + 660 with a total of 910,042 "DYWIDAG" concrete ties from Km. I-395 + 225 to I-406 + 716 and a total of 22,034 Bi-Block concrete ties with a length of 561 + 492 km. of track. This track was constructed between 1972 to 1973.

4.-Division Jalapa Line "V"

Between the kilometers V-468 + 186 to V-335 + 959 and with a length of 132 + 227 km. of track and with a total of 165,580 "DYWIDAG" concrete ties. This track was constructed between 1974 to 1975.

5.- Division Mexico Line "A"

Between the kilometers A-0 + 700 to A-23 + 474 and B-0 + 700 to B-24 + 100, as well from km. H-0 + 000 to H-19 + 000 with a length of 55 + 154 km. of track and a total of 91,802 "DYWIDAG" concrete ties. This track was constructed between 1967 to 1968; lines "A" and "B" and Line "H" in 1984.

6.- Division Mexicano Line "S" and "SH"

Between kms. S-0 + 000 to S-59 + 550 with a length of 59 + 550 kms. of track and with a total of 85,008 "DYWIDAG" concrete ties, as well from km SH-0 + 000 to SH-22 + 958 with a length of 29 km. and a total of 48,314 "DYWIDAG" concrete ties. This track was constructed between 1970 and 1977.

7. Division Monclova Line "R"

Between kms. RA-79 + 200 to RA-118 + 000 with a length of 39 + 600 kms. of track with a total of 63,360 concrete ties type "RS" and in line "RD" from km. 177 + 556 to 216 + 992 with a length of 39 + 366 kms. of track and with a total of 64,284 "DYWIDAG" concrete ties. This track was constructed between 1971 and 1972.

8. Division Monterrey Lines "B" And "F"

Between kms. B-998 + 100 to B-1290 + 000 with a length of 291.9 km. of track and with a total of 441,242 "DYWIDAG" concrete ties. We used this same tie between km. F-53 + 0 to F-178 + 600, with a total of 281,554 ties in Line "F" between km. F-9 + 000 to F-53 + 000. There is tie type "RS" with a total of 124,097. This track was constructed between 1971 and 1975.

9. Division Pacifico Line "NE"

Between kms. NE-0 + 000 to NE-193 + 831 with a length of 193 + 831 kms. of track and with a total of 322,922 "Bi-Block" concrete ties. This track was constructed in 1980.

10. Division Queretaro Lines "A" And "B"

Between kms. A-23 + 454 to A-46 + 500 and from 206 + 240 to 215 + 730 there are a total of 75,226 "DYWIDAG" concrete ties. This total line was constructed between 1970 to 1972.

10 A.- Lines "AQ" And "BQ"

With the construction of the double railway Mexico-Queretaro in the charge of Secretaria De Comercios y Transportes in Line "AQ" and "BQ", there have been 173,545 "DYWIDAG" concrete ties installed as well as "RS" and "SL" with a length of 104 + 169 kms. of track. It is actually operating in Division "AQ" between Teoloyucan, Mex. and Ahorcado, Qro.

11. Division San Luis Line "B"

Between kms. B-485 + 720 to B-522 + 480 and from km. B-840 + 000 to B-915 + 479, with a total of 139,619 "DYWIDAG" concrete ties and a length of 66 + 600 kms. of track. This work was done between 1967 to 1972.

Line "BC"

Between kms. BC-0 + 000 to BC-180 + 143 with a total of 299,023 "RS" and "SC" type ties with a length of 180 + 000 kms. of track. This track was constructed by Secretaria De Comunicaciones Y Transportes and given to the railroad on 1972.

12.- Division SE-VCI Line "G"

Between kms. G-102 + 500 to G-127 + 000 with a total of 37,084 "DYWIDAG" concrete ties a length of 24,500 kms. of track. This track was constructed in 1964.

13.- Division Tenosque Line "FA"

Between kms. FA-483 + 000 to FA-493 + 000 with a total of 16,600 "DYWIDAG" concrete ties and a length of 10 + 000 kms. of track. This work was done by the Ex-Ferrocarriles Unidos Del Sureste (FUS) and actually administrated by the Mexican National Railways. This track was constructed in 1979.

The Ferrocarril Del Pacifico has 67,177 "DYWIDAG" concrete ties; the Ferrocarril Chihuahua-Pacifico a total of 726,725 Bi-Block ties, and the Sonora Baja-California Railways has a total of 167,600 "DYWIDAG" concrete ties.

We will continue to use concrete ties in our railway system our railway system with modern track, and having an extension of 3,470 kms., we will continue with this effect on all the lines that need it, for the purpose of upgrading the quality of the track, giving more safety, comfort and speed to our train traffic and offering better service to our customers.

BRIDGE ERECTION ON LINES WITH TRAFFIC

Ing. G. Rivera D.

1) General Information

The Ferrocarriles Nacionales de Mexico system was built mainly in the last century and in the beginning of this century. Both the track as well as the bridges were designed with the requirements of the time for rolling loads considerably lower than those at present.

With the passing of the years and the development of the rolling stock, tracks have been upgraded to be able to comply with their function, reinforcing embankments, changing rails, increasing progressively its size, substituting ties, ballasting etc. The bridges were left in their original conditions. No improvments were made to these structures to withstand the increase in live loads. They were the weak points in the rail tracks.

2) Rolling Loads

The traction in our system is based on diesel electric locomotives. We operate with mainly two types of locomotives on the main tracks. These locomotives are the B-B of four axles, 2000 H.P., with a maximum weight of 120.3 metric tons and the C-C of six axles, 3000 H.P., with a maximum weight of 173.73 metric tons. In reference to freight cars, there is a strong requirement for the movement of cars of 100 ton capacity from the ports of entry at our northern border to the interior of the country. These cars transport mainly grains in hopper cars, as well as chemical products in tank cars. The gross

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weight of these units is 119.5 metric tons. It has been calculated that the effects that are produced by the actual trains with these locomotives and freight cars of 100 tons capacity is, in general, lower of E-66 for short spans (5') less than E-55 for medium size spans (50'). For bigger spans the effect is less.

The goal to be accomplished is to standardize the load capacity of the bridges to a Cooper E-72 load to cover actual needs and provide a good margin for future traffic.

3) Existing Bridges

The existing number of bridges in the system is 10,200 with a total length of 110,040 mts. Of this total 3,850 are wooden bridges; 5,400 are steel bridges and 950 are built of concrete.

The load capacity of the bridges in the system is varied, depending on the time in which the tracks were built, its importance or its purpose. For the record, as examples, many of the bridges of the Mexico-Nuevo Laredo line are of E-60 capacity. The main capacity of the bridges in the Mexico-Ciudad Juárez is E-55 as are those of the Guadalajara-Manzanillo line. On the Transisthmian, the general capacity is a E-40, and in the line from Veracruz to Istmo de Tehuantepec it is E-36. There are some lines of lower capacities; for example, the Ixtepec-Tapachula to the Guatemala border has many bridges which are E-30.

The theoretical capacity of the steel bridges can be increased, taking in consideration that the impact loads produced by the diesel-electric locomotives presently used are smaller than those of the steam locomotives, and that the bridges were designed for this last type of traction.

The increase depends on the type and the span of the structures and is between 10% and 17%. However, the capacity of the bridges diminishes with time due to weardown, corrosion and fatigue; and the decay factor depends on the physical condition of the bridge.

In the case of wooden bridges where no increase in capacity is taken into consideration due on reduction of impact, deterioration has to be taken in consideration due to the effects of wood rotting fungi and wood eating organisms.

4) Existing Types Of Structures

4.1 Substructure

The wooden substructures are trestles on wooden piles in accord with the AREA type projects either with driven piles or supported.

The majority of substructures are abutments or piers of the gravity type built of masonry or of cyclopean concrete, and in some cases with reinforced concrete caps and in other cases of dressed stones or plain concrete. In bridges with big spans, the piers and abutments are generally built of concrete-driven cylinders.

The substructures built with reinforced concrete or concrete piles are either reinforced or prestressed and appear only in bridges of relatively recent construction.

Substructures of steel trestles in viaducts of considerable height also exist.

4.2 Superstructure

The most common superstructures for bridges of small spans are girders, rails, or wooden longitudinal stringers. In median size spans, the most usual type is a steel open deck girder consisting of riveted or rolled beams. Also used frequently in these median size spans are steel through girders with the main girders riveted and floor beams and stringers with sections either riveted or rolled.

For bigger spans the superstructures are formed by trusses open deck or through type riveted or articulated. The use of welding elements are of recent times.

The reinforced or prestressed concrete girders belong to very recent construction.

5) Adopted Solution

In 1972 a program was started to increase the capacity of the basic system to Cooper E-72 capacity to enable the operation of heavy trains without restrictions. To obtain this increase in capacity, two procedures have been followed:

1) Reinforcing the existing structures, and
2) Substitution of the structure.

Reinforcements are made in cases where it is more economical than the substitution. This depends on the original capacity of the structure and the physical conditions.

In this talk we are referring to the substitution of bridges and in particular of the superstructures in the tracks with normal operation of trains.

In the last years more than 1,500 substitutions of bridges have been made. But the pending work to be accomplished is very hard. To be able to comply with it during this Administration, investments for this purpose have to be sharply increased.

In general, it must be pointed out that substitution of an old bridge by a new bridge is made in the same location. The most important problem to resolve is to keep established traffic movements.

The replacement of the superstructure makes it necessary to first eliminate or tow the old structure to be followed by the erection of the new one, maintaining the maximum possible speed in these two operations to avoid the alteration of timetables.

There are a few occasions when replacing a bridge a change in location is made to improve the layout of the track. In these cases the problem of construction is simplified because sufficient time is available and revenue traffic does not have to be rerouted.

6) Erections

As the replacement of superstructures is made with the tracks in operation with train traffic, in a general way, we are dealing with prefabricated steel structures or concrete structures that have been built in places other than their final location. To be able to set and integrate the structure in its final working location, it is necessary to make the erection of the same.

It is very difficult to make a radical classification of the procedures of erection due to the fact that they are numerous and varied with the characteristic properties of each work; but for the purpose of having a reference frame, we will make general classifications, dividing the procedures into three general categories.

- 6.1 Erection of structures assembling each part individually at the final sight of the work,
- 6.11 Erection or assembly of the structure near the work and its final setting by means of a limited movement, or
- 6.111 Total construction in the shop, transportation from great distance, and erection by means of machinery or hoisting.

6.1 The first process has the advantage that it does not require work with hevy duty elements but has the inconvenience of occupying the track for long periods of time. For this process a false framework must be built that will support the structure that is being assembled and at the same time is able to uphold the transit of the trains during the time that the erection is taking place.

6.11 The second process is used when the superstructure is being built or assembled near the final location, either aligned with the new work or parallel to same and by means of a longitudinal maneuver or launching is put in its place or by means of a lateral displacement is taken to its final location.

The maneuver of launching is only used in bridge constructions in locations other than where the original bridge exists. Generally a false framework is not needed. In this maneuver it is necessary to consider the reversal of stress that appears during the process. The maneuver of the lateral displacement is indicated in lines with traffic because the interruption of this is limited only to the displacement time. For this lateral displacement a false framework is required to support the new structure and receive the old one. The displacement of the new structure and of the old structure can be simultaneous, or the old structure can be moved to clear the space that will be occupied later by the new structure.

6.111 The third process implies that the structure will be built generally far away from its final destination, that transportation, hoisting and the handling of the heavy elements with machinery are necessary.

The transportation can be terrestrial, maritime or fluvial and the machinery and cranes on crawler tread, on pneumatic tires, on railroad trucks, and cranes or booms on barges or special gantries.

This process does not need a false framework but needs the proper machinery to be able to handle the weight of the superstructure to be erected.

7) Election Of The Type Of Erection

The election of the system of erection of a bridge generally is based on experience with similar cases applying different variants in the system that are the product of the ideas of the man responsible for the erection.

To determine the erection process the following considerations have to be studied:

7.1) Economic consideration of the erection.
7.2) Time availability for erection.
7.3) Availability of equipment.
7.4) Type of structure to be erected.
7.5) Characteristics of the span to be cleared.

7.1 Economic Considerations

As in all engineering work economic considerations are basic. If in a work it is feasible to apply different processes, each one has to be analyzed and the most economic will be selected, taking in consideration all the factors that form part of the cost. Not only the cost of the maneuver but the consequential effects caused by the interruption of the traffic in the line must be considered.

7.2 Time Availability

The available time for the erection is decretory for the election of the process.

The case of erection on lines with traffic is completely different from the erection of a structure in a new line or of a sidetrack. On a track with traffic the time given for the track occupancy of the main line is normally of some hours, depending on the traffic density. It is very important, therefore, to formulate a program of erection, which must include all the activities of the maneuvers, even taking into consideration the time length in minutes of each one. Special care in the estimates of time of each activity has to be taken with no shortcoming of any of them. If there are shortcomings or wrong estimates, the authorized time for the erection can be overpassed thus causing delays in the operation of all the system.

It is recommended that in all cases a thorough review of the program of erection be made before making a final decision on one of the processes.

7.3 Available Equipment

The availability of equipment is determinant for the election of the process of erection.

With high-capacity cranes or "derricks" the times of maneuver can be greatly reduced during the erection of big girders. When the capacity of the equipment is not sufficient, the erection has to be made in parts.

A basic datum is the length of the boom of the cranes required to perform the erection. In some cases the available cranes are of such great capacity but with a short boom that their total capacity is not used.

Previous to the establishment of the process a verification of the capacity of cranes in accordance with the diagrams of radius and swing operating freely and with blocking is necessary to select the way that the cranes will operate.

In our system which is limited in resources, suitable equipment for erection is not always available. The N de M has cranes of 40 ton capacity mounted on railroad trucks and a few cranes mounted on pneumatic tires or crawler treads. Sometimes we can use wrecking cranes, which are used for rescue of accidents and which are not the proper type for erection because of their short booms.

7.4 Type Of Structure

The type of structure depends basically on the span to be cleared and is determinant of the process of erection to be used. The degree of difficulty increases as the span is bigger. Also to be taken into account are the types of joints that integrate the structure. Because of the time of assembly the most adequate process can be decided to be used.

In structures that will be assembled in sites of the application it is important to verify that the joints match to avoid last minute fittings or modifications. All the details of this joint must be checked from the moment of the manufacture of the structure.

In respect to the structures project, it is necessary that from the moment of the formulation the method of installation to be considered should not be carried out independently from the erection. The erection is a stage of the structures life, and must be considered from the beginning of its engineering.

7.5 Characteristics Of The Span

The topographic and hydraulic characteristics are important factors to be considered for the election of the process of erection. If the rivers are dry and not deep, the construction of false frameworks will be of advantage. This is not so in deep rivers or those with permanent water.

8) Most Frequent Cases Of Erection In The National Railroads

8.1 Erection Of A Concrete Structure

The concrete superstructures most commonly used are monolithic slabs of reinforced concrete for small spans up to 7 m. (23'); for medium size spans, up to 15 m. (50'), prestressed concrete girders of the box type, formed by 4 beams joined by means of transversal post tension.

The most common substitution is to eliminate wooden trestles and wooden girders, or rails skided.

The substitution is accomplished following the process 6.111 with a 40 ton crane.

In these cases, which are the simplest, a working program is formulated, considering the following steps.

A. Withdrawal Of The Track

The dismantling of the track is generally made in modules, with the ties attached to the rails. The joint bars have already been eliminated. When dealing with welded rails, the rails are cut in the bridge span and are changed to rails with joint bars.

B. Dismantling Of The Superstructure

By means of a crane the old girders of the bridge and all the elements of the substructure that are in the way of the new superstructure are removed.

C. Erection Of The New Superstructure

By means of a crane the new girder is hoisted by the steel handles that are left for this purpose in the slabs or girders. The new superstructure is brought generally from a place near to the bridge and placed at the side of the track on the caps or crossheads of the new bridge. For this operation a crane of larger free capacity than the weight it is going to move in the resulting radius is used to transport the structure. After the erection of the new superstructure, the transverse connectors are placed if the structure is formed of girders of a box section.

D. Restoring The Track

The original track with the same ties can be put back, or new ties may be used. Meanwhile, the ballast is put in and the track is blocked on wooden blocks.

At the moment the track is interconnected, the flow of trains is reestablished with slow orders which limits the speed to 10 Km/hour.

The normal speed of operation is reestablished as soon as the construction of the track, ballast, alignment and leveling is finished.

When we have a bridge with many spans, the process of erection is basically the same. If we use 2 cranes, the time of maneuver can be reduced using one crane for dismantling and the other for erection.

If the erection is going to be made in a viaduct where the access of the cranes to the riverbed is possible, it is advantageous to work with cranes mounted on pneumatic tires, thus reducing the time of maneuver.

8.2 Erection Of Steel Superstructures

For spans less than 30 m. (100') it is common to use metallic girders of solid web. These girders can be either of the open deck type or through girders.

For spans bigger than 30 m., the use of steel open deck trusses or through trusses is very common.

8.2.1 Girders Of Solid Web Open Deck Type

In the erection of these girders the procedure is the same as in the case of the concrete griders, erecting all the girder in one operation if the spans are short and the weight can be managed by one crane. In the case of big spans on the order of 30 m., the procedure is made with two cranes, hoisting the girder by both ends, blocking the cranes and hoisting the structure in a parallel position with respect to the final location of same, where it has been placed previously. If ihe substitution to be made is of an old metallic girder, first the span is cleared eliminating the girder with two cranes and placing it on a false framework at a side of the track. It is very important to check that the cables of the slings and of the crane are in good working condition and their capacity is enough to handle the load, taking in consideration the weight to be handled. Generally a safety factor of 4 is considered.

8.2.2 Steel Through Girders

The most used process in this type of superstructure is the 6.11, limited travel, for which reason a transversal shifting is made supporting the new girder over a false framework and performing the launching pushing with jacks. This process is used when the weight of the structure exceeds the crane capacity.

8.2.3 Trusses

In many cases we have made the substitution of steel through trusses.

The trusses generally are welded members and field connections riveted. During erection interference with traffic has been kept to a minimum, reducing the period of time the track is occupied.

The old process used for the substitution of frames is the following:

a) Construction of a false framework for supporting the old truss.
b) Dismantling of the truss blades, disconnecting member by member and retaining them.
c) Elimination of the floor system, floor beams and stringers.

d) Setting of the floor system of the new truss over the false framework.
e) Erection member by member of the blades of the new frame as well as portals and top lateral bracing.
f) Elimination of the false framework.

This process has the disadvantage of greatly obstructing the traffic because the dismantling of the old frame as well as the erection of the new member by member it is necessary to work over the main track with one or two cranes for long and numerous periods of time.

The process actually in use and with which more than 30 trusses have been changed is the following.

A) Construction of false framework to support the floor system of the old truss.
B) Disconnection of the floor system with respect to the old truss blades. Withdrawal of the old truss extracting the two blades as well as the top lateral bracing, leaving the floor system over the false framework to be able to support the traffic.
C) Placement of the new truss assembled with the two blades, portals and the top lateral bracing in the place where the old truss was located.
D) Change of the old floor system by the new one.
E) Riveting the connections of the new system of the floor with the truss. Elimination of the false framework.

The false framework most used consists of trestles made of wooden piles and steel girders which cover several panels of the truss. This false framework is specially recommended when the number of panels of the old truss and the new one do not match. In this manner there are enough free spaces that are sufficient to allow the flow of the river.

A) The elimination of the rivets in the connections of the floor beams with the truss is carried out several days before the erection substituting rivets with bolts. One day before the maneuver the bolts are taken out thus leaving the frame completely separated from the floor system.
B) The withdrawal of the old truss is made in one operation hoisting the truss with its portals by means of two cranes that span the track, hauling same out of the bridge using the traction of the cranes. The old truss is placed in an auxiliary track where it can be stripped easily to be used in a secondary line. The placement of the new truss is carried out in the same way as the withdrawal of the old. The new frame is previously assembled and is placed in a track near the bridge without obstructing the train traffic. With two 40 ton cranes, a truss of 36 m. (120') can be managed. For trusses up to 48 m. (160') the erection is made with a wrecking crane on one end and 2 cranes on the other end. In frames of 60 m. (200'), not having cranes with sufficient free capacity, the use of pneumatic jacks have been implemented together with railroad flat cars instead of cranes.
C) The change of the old floor system is made by means of two cranes, one dismantling the old longitudinal beams and the cross beams and the other putting the new ones with the same process of erection as the solid web girders.
D) The connection of the floor system to the truss is a routine operation and is made without obstructing traffic.
E) The withdrawal of the false framework is programmed in free time of traffic once the new truss has been totally integrated.

With this process the maximum times of occupation of the mainline are reduced in trusses up to 36 mts. a time of two hours and four hours for trusses up to 48 mts. In frames of 60 mts. the time used is approximately six hours.

When dealing with a bridge of various spans with different types of superstructures, the erection is made combining the processes.

We have tried to systematize the erection works by implementing processes of bridge erection and taking into account the shortage of resources and especially the lack of equipment that these railroads have. The lack of the suitable machinery has been surpassed with the effort and the good will of the crews. Due to this, all the erections have been successful, complying with the authorized timetables without causing interruptions or delays in traffic and without any regrettable accidents in more than 1,500 erections executed.

Design and Manufacture of the Concrete DYWIDAG Tie

Ing. R. Rizo H.*

The prestressed concrete "DYWIDAG" tie is a patented design of Dyckerhoff and Widman Company of Germany, based in a concrete of zero shrinkage with characteristics in accordance with D.I.N. standards in cubes of 8" side, with a compression resistance of 8530 lbs/in^2 at 28 days and to 925 lbs/in^2 at 7 days.

This prestressed system is post-tensioned by means of a high carbon steel wire, cold drawn, whose yield point is at 14,000 kg/cm^2 (199,000 lbs/in^2) and a breaking point of 16,000 Kg/cm^2 (227,000 lbs/in^2). It is manufactured in a hairpin form with rolled/threads at the ends of the hairpin and anchored by means of a conic nut-lock that rests in belled anchors which are built in the concrete for this purpose.

For the structural design, we take in consideration the track characteristics, the size of the rails, the maximum axle load, the dynamic speeds of operation and the impacts or overloads produced by the rolling stock.

The determination of the loads that are applied to the tie will depend basically on the result of the combination of values of influence shown by the mentioned factors.

A practical method to evaluate the load value that is transferred to the tie was developed by Prof. Eisenman of the Technical University of Munich by means of prototypes developed in the laboratory and direct measurements in the tracks that analyze the deformation of the rails like a continuous elastic beam. This beam is supported over springs that simulate the deformation of the ballast under the application of a determined load distribution.

The rail deformation is analyzed and takes into account a series of values that depend on the ground condition, the separation between ties, the static load application for a tie in the worst conditions which depends on the speed of operation, the real condition of the track, the frequency of trains and rolling stock conditions. An impact factor is applied to increase the static load to obtain the dynamic load effect.

In a similar way, the A.R.E.A. gives a series of recommendations to obtain a value of the load distribution factor per axle in function of the separation between ties and considering an increase of 150% of the static load for the impact.

However, this value depends specifically on each case particularly for a determined track. Therefore, it is recommended that A.R.E.A. make a series of determinations to obtain more realistic results in the tracks.

In Pueblo, Colorado, under well controlled conditions, a series of values were obtained that in some cases are below the real conditions of the track.

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The reaction of the ballast support will be a function of the track compaction frequency for a determined traffic and the ballast thickness between the tie and the embankment as well as the soil condition or its compaction. That also will be a specific condition of each track. Once the load is determined and values that will be supported by the tie are obtained, the analysis of different sections is made to obtain the resistance moments as a function of a proposed geometry. The effective load and eccentricity of the prestressing for different values of concrete stresses take into account that the tie is an item subject to flexure and shearing stress under the loads that are transmitted by the rails, and a reaction of the support over the ballast determines the moments diagram.

Table 1 shows different types of ties varying basically in the positive resistance moments under the rail seat. They have basically the same resistance for the negative moments in the center of the tie. This is in accord with the Mexican experience, and gives us the means to select the proper type of tie in accord with the different conditions of track, equipment, speed, etc.

POSSTTENSIONED CONCRETE TIES DYWIDAG RESISTEN MOMENTS

DURMIENTE TIPO	LARGO	PESO	ASIENTO DEL RIEL (+)	ASIENTO DEL RIEL (—)	CENTRO (—)	CENTRO (+)
TYPE TIE	LENGTH	WEIGHT	RAIL SEAT (+)	RAIL SEAT (—)	CENTER (—)	CENTER (+)
B58	2.40 m.	247 kg.	175 Ton.-cm.	95 Ton.-cm.	158 Ton.-cm.	79 Ton-cm
	8'	544 lbs.	152 Inch-Kips	82 Inch.-kips.	137 Inch.-kips.	68 Inch.-kips.
B58-FM.	2.40 m.	275 kg.	240 Ton.-cm.	125 Ton-cm.	210 Ton.-cm.	115 Ton.-cm.
	8'	606 lbs.	208 Inch.-kips	115 Inch.-kips	182 Inch.-kips.	100 Inch.-kips
I-84	2.40 m.	300 kg.	253 Ton.-cm	132 Ton.-cm.	228 Ton.-cm.	104 Ton.-cm.
	8'	861 lbs.	220 Inch-kips.	144 Inch.-kips.	200 Inch.-kips.	90 Inch.-kips
I-85	2.50 m.	330 kg	300 Ton-cm	184 Ton.-cm.	200 Ton.-cm.	138 Ton.-cm
	8' 3"	727 lbs	260 Inch.-kips.	160 Inch.-kips.	175 Inch.-kips.	120 Inch.-kips.

TABLE 1

Manufacturing Process

The process starts with the storage of the aggregates, which have been previously classified and washed. The aggregates have a high resistance to compression, are free from slime and organic material, have cement characteristics Type III for high and quick strength and the water used is pure as potable water. With these elements we proceed to manufacture the concrete by mixtures that have been previously mixed and controlled by weight to obtain uniformity and homogeneity.

Each mixture of zero shrinkage with a maximum aggregate of 1-½" is used to manufacture 2 ties that will be poured in individual molds in groups of 6 units for each production line working in couples, 2 in the assembly or preparation process, 2 in the pouring process and 2 in the immediate demolding, working simultaneously and continuously to give immediate rotation to the use of the molds.

For the assembly, the anchoring of the fastening is fixed in a determined position, and the ones for the prestressing and the ducts for the steel, afterwards.

In the pouring process the compaction is accomplished by the high frequency contact vibrators and by a plate with normal frequency vibrators that allow the arrangement of the concrete in reduced time to have a production of 2 ties every 3 minutes for each production line.

The demolding is made immediately and the fresh ties are put on a pallet that allows them to be handled without causing any deformation and to stow them for a steam strengthening process.

Each fresh tie is checked for its dimensions and stowed in groups of 40, which is the equivalent of one hour of production, to form humid steam strengthened groups with gradually controlled temperature during 8 hours.

Each strengthened tie is transferred to a resting area where they are kept for a period ranging from 3 to 7 days to obtain a minimum concrete strength of 450 Kg/cm^2 (6400 lbs/in^2) before prestressing the tie.

The applying of the initial prestress load is accomplished by means of mechanized hydraulic jacks coupled to equipment designed designed to allow stabilizing the load in a uniform way to efficiently transmit the prestressing to each tie.

Once the prestressing is finished, the ducts are sealed under pressure with whitewash to protect the steel against rust, the ends are sealed with mortar, and the tie heads are waterproofed.

The finished ties are stored until they achieve the final strength on the 28th day and can be dispatched on railroad flatcars to their location for installation in track.

Quality Control

The quality control is established as a continuous process in each of the components of the tie, as well as the humidity determination of the sand, the graduation of each of the aggregates, the pureness of the cement and its mechanical properties of strength, the daily strength test of the concrete on different times of compression and flexion on the 7th day, mechanical test of compression and whitewash, and the comparison of the graphics from the steel deformation and the flexion of the tie.

By this manufacturing process, more than 60 million ties have been produced worldwide, and more than 6 million in Mexico during 17 years that are located throughout the Mexican railroads. Approximately half a million have been exported to Venezuela for different railroads in that country.

Rail-Tie Elastic Fastening

Ing. A. Diaz A.*

The birth of the railroad as a massive transport medium had a great impact in the world. Among other things we can consider that it was the basis for the industrial revolution of the 19th century.

In it's beginning, railroad track was built with rails, wood ties and iron spikes as fastenings to keep the right gauge. To join the rails, joint bars bolts and nuts were used.

In regard to tractive and rolling equipment, we can say they were both very light on the early railroads, and the speed they developed was very low. The technical advances made during the last years of the past century and the beginning of the present one made it possible to build heavier equipment that could handle greater volumes of freight at faster speed.

On the other hand, the advances attained by the steel industry made it possible to manufacture heavier and longer rails, thus considerably improving the strength and quality of the track. A great advance was obtained by the use of anchors and continuous welded rail.

After these improvements, railroad technicians were compelled to project better and stronger tracks, capable of supporting the safe passage of increasing loads hauled at ever increasing speeds.

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The technical modifications to tracks were directed mainly at improving the geometrical specifications, both on the vertical and the horizontal plane. Regarding the structural reinforcement of track, it was obtained by the use of heavier and longer rails up to 140 pounds per yard (75.62 Kg/m).

On the construction of new railroad lines in Mexico 115 pound rail is being laid now in 39 foot lengths.

The use of tie plates became general. This allowed a better distribution of stresses on the wood ties, thus considerably increasing the useful life of ties.

We can well consider that the construction techniques of railroad track were not changed until the first half of this century. Other activities of the railroad industry were in full development, changing and improving equipment and facilities in order to keep up to date with the new advances of railroad technology. We can name as examples: the appearance of diesel-electric and electric locomotives which changed the motive power area; the heavier and more sophisticated rolling stock; and the communication and signaling systems which have caused very important reactions to the design and operation of railroads.

As a result of that, the stresses on the track structure increased considerably, especially axle loads with their vertical and lateral stresses, impact stresses and vibrations, as well as the deformation the track receives with the passage of heavy trains.

As a result of this, track maintenance costs soared alarmingly, compelling track engineers to look for substantial improvements in track structure.

After the World War II, many important changes in railroad track appeared. I will here mention the main ones:

a) The use of long welded rails, by either the electric or aluminothermic processes, which eliminated the joints of rails every 33 or 39 feet, and originated irregularities of track that shortened the useful life of rails and endangered the train movements.

b) The extensive use of concrete ties that gave a greater stability to the track and also helped to solve the growing problem of getting (in local or international markets) wood ties which had become increasingly scarce. For a long time attention had been directed towards a substitute industrial tie capable of meeting the technical and economic requirements.

c) The elastic fastenings that offer the adequate fastening between rail and ties.

The appropriate solution of these changes has resulted in the modernization of the main railroad systems of the world.

As good examples of modernization of railroad track we can name Japan, with its new Tokaido line, where the bullet train and modern track has permitted it's trains to reach a commercial speed of 200 Kms. per hour. And France has lately inaugurated its new line Paris-Lyon line, with commercial speed above 250 Kms. per hour. It is worth mentioning the efforts and results that several other countries, such as the Economic European Community, Canada, United States, Brazil and Mexico, have obtained in modernizing their railroad lines.

It must be made clear that in those countries where no efforts have been made to modernize the railroads, their economic and social importance have been slowly lost, yielding to other transport modes, such as trucks for freight haulage and to planes for passenger movements.

Considering that in the present meeting other speakers will amply refer to diverse specific themes, I will only talk about rail tie fixation, since Industrias NYLBO, S.A., División Nylco Mexicana, which I represent, manufactures in Mexico the fastenings used by Mexican railways. The assembly of rails (short or long), ties (wood or concrete) and elastic fastening is called "Elastic Track", precisely by acknowledging the particular characteristics that this type of fastening provides.

Mexican railroad companies started the use of elastic track at the beginning of the sixties. The increasing difficulties of obtaining wood ties was alarmingly deteriorating the quality of it's tracks, dangerously increasing the wood tie deficit for normal maintenance and at the same time delaying new track construction. These tracks were needed to complete the railroad system and fulfill Federal Government programs.

With the above in mind it was necessary to find the means to solve such a serious problem. The use of concrete ties on the railroad system as a substitute or as a complement to wood ties was the first major step taken.

To succeed in the use of concrete ties, a group of Mexican railroad technicians studied the problem, including trips to those countries who had installed concrete ties on their tracks in a massive way.

During those years European countries, especially France, Germany, England and Sweden, were the leading countries in that respect.

As a basis for a final decision, the following premises were considered.

a) The technical solution of rail tie fastening should be reliable, economic and lasting.

b) It should be possible to manufacture in Mexico.

c) It should be used on any type of concrete tie.

d) The same basic fastening system of rail to tie could be also used on wood ties.

From these studies the conclusion reached was to adopt a fastening usable both on concrete or wood ties.

This elastic fastening for concrete ties is made up of 2 elastic clips, 2 T-bolts with nuts, one elastomer grooved pad and a shock absorbing element on the exterior side of each rail.

The elastic clip is made so that the top branch furnishes the rail-tie tightening. The lower branch furnishes the lateral butt of the rail base and being elastic too in this direction helps greatly to lessen the lateral impacts. The buckle between the two branches transmits and distributes directly to the concrete the lateral stresses generated by the run of trains.

The clip is so designed that by eliminating the separation between the top branch and the rail base, the anticipated tightening is obtained as a result of the elastic deformation.

The tightening load of the clip, when the above-mentioned separation is nullified, and as a result of reducing more than half the lever arm between the bolt and the point of support on the rail base, allows an overload from 100% to 150% with very insignificant deformation. This permits the assimilation of extraordinary stresses counteracting the rails' turnover trend.

We have applied loads up to 5 tons during laboratory tests losing only 20% of the original tightening as the load is retired. This characteristic of the clip is very effective in preventing rail turning.

The absorption of the transversal dynamic actions toward the exterior of the track and concrete protection are increased by a shock absorbing element placed between the clip's buckle and its seat on the tie.

A great economy is obtained on the building cost of railroad lines by the elimination of steel tie plates.

The assembly design allows its use on different rail sections on the same tie design. The track gauge can be easily changed, if necessary, by only changing the clip and using the other parts of the fastening.

This is possible because the clip design contemplates changes in the length of both branches, keeping the buckle and the place of the bolt hole constant on the tie.

The great versatility of this elastic fastening permits the standardization of railroad lines, with a great reduction of maintenance and rehabilitation cost because it permits the use of different rail sizes on the same concrete tie type. This makes possible the recovery and the reuse of this very expensive and long lasting part of the railroad track.

As an added benefit we can mention that the technical conditions of the track do not change. In this way it is not necessary for the workers to be learning new ways of maintenance or rehabilitation.

It is very important to mention that the basic components of an insulated or non-insulated track are the same. In the first case an insulated element that fits easily in the other components and is not subject to wear is adapted. So it is very easy and economical to turn a non-insulated track into an insulated one.

All the above results in very important economies when tracks are built, rehabilitated, maintained or modified.

The T-bolt is the part that holds the clip, so that this transmits the tightening to the rail. On account of it being an adjustable joint and its fitting capacity, it compensates very well for admissible variations from the nominal dimensions of the different parts (rail, tie, clip, elastomer pad and the bolt itself) allowing a tightening load which is reasonably uniform.

T-bolts have a mark on the top, easily seen, which shows the correct position of the bolt.

It is very important to install a steel washer between the nut and the clip, since this washer prevents any damage to the clip surface during tightening that may impair the clip's resistance to fatigue, and distributes the load over a greater surface of the insulating washer when this is used.

The grooved elastomer pad that goes between the tie and the rail is 4.5 mm. thick, with a width and length in accordance to the rail and tie to insure the right setting of the rail.

The half cylinder rubber element that supports the buckle of the exterior clips is 85 mm. wide and 4 mm. thick. NYLCO has developed a celoron element that has proved technically better than the rubber pad.

The insulated celoron washer (bakelited canvas) is a cylinder with a 60 mm. diameter, 5 mm. thick and a central 23 mm. hole that lets the T-bolt go through. This is only used on insulated tracks for signaling.

This fastening system has the advantage that with a relatively small investment and in an easy way can be reinforced when necessary. For this purpose an additional spring steel leaf is placed between the top branch of the clip and the steel washer.

Together the clip and the elastomer pad join, without any looseness and in an elastic way, the rail to the tie in such a way that the following advantages are obtained.

a) Keep the gauge and to restrain longitudinal movements of the rail.

b) Distribute in a more effective way the load rail-tie.

c) To diminish impacts between rail and tie.

d) To act as a filter for vibrations.

We found that this fastening system presents problems on curves with a radius smaller than 380 m. Based on field tests, several alternatives have been studied, and by using the already installed concrete ties may solve the difficulty by only changing the exterior clips.

The results obtained in the comparative laboratory test carried out in accordance with AREA concrete tie specifications, Chapter 10, Articles 1.9.1.11 and 1.9.1.13 (Test With Repeated Loads and Lateral Resistance respectively) have been very satisfactory.

At the end of this year we shall be able to present our customers the prototypes for the corresponding field tests.

This basic elastic fastening system is also used on wood ties, keeping the same fundamental rule of high resistance to overloads that may induce rail turning beside the basic function of providing the necessary tightening between rail and tie to avoid longitudinal movements.

The fastening is very economical on tangents or low degree curves (less than 2°, 573 mts. radius), since in such cases it is not necessary to use steel tie plates. The elements per tie are: 4 screw spikes, 4 clips, 4 grooved steel plates, and 2 elastomer grooved pads. The work of the grooved steel plate is to distribute the load transmitted by the clip over the wood tie and to prevent the clip from turning in the horizontal plane.

It must be mentioned that when using this fastening, it is necessary to previously groove and bore the wood ties. This has the advantage of securing the right gauge, simplifying the track laying.

When the curve radius is less than 573 mts. it is necessary to use steel tie plates to assimilate the strong lateral forces. The number of screw spikes and clips must be increased to six per tie.

Screw spikes are manufactured in two different sizes. One is used on tangent track, and the other when using base plates. In both cases the same length of grasp in the tie is maintained.

Both screw spike types are made from one piece without any welding. The head is made by a hot heading and the deep threads of the screw spike are obtained by the hot rolling method. The head is square and the screw spike diameter is 23 mm.

A particular characteristic of the screw spike needed for the right performance of the system is the depth of thread which insures very high resistance to extraction. This resistance to extraction is 50% higher than that of screw spikes with normal thread.

The metallic tie plates for rail used on elastic track on wood ties for curves higher than 2° are made of A-36 steel, 12.5 mm. thick (1/2") with a length of 285 mm. and a width of 197 mm.

They have notches on the edge parallel to the rail for the proper positioning of the elastic clip and to keep these from turning, they have 3 circular holes that permit necessary screw spikes to go through. They have 3 welded square pieces, 12.5 mm. each side, that keep the track gauge and resists the lateral loads improved by the passing trains.

Industrias NYLBO, S. A., División Nylco Mexicana started operations in 1968 as the first specialized plant in the country to manufacture railroad track fastenings.

Located in Ecatepec, the state of Mexico's industrial zone, the plant has a processing capacity of 9,000 tons of steel a year. Fifty percent of that amount goes to the lines producing elastic track fastenings and the remainder to the lines producing conventional fastenings. NYLBO also produces a complete range of coil springs for railroad cars.

We have furnished to date railroad fastenings covering 100% of the national needs plus some for export. We have also produced more than 21,000 tons of track spikes and 10,000,000 anchors for regular track. For the elastic fastening system over concrete ties, we have manufactured more than 27,500,000 sets. We have provided more than 2,500,000 pieces of helicoidal springs for railroad cars.

We must mention that we get our raw material in the national markets. This added to our plant and equipment, qualified labor, supervisory personnel, management and technical people means total integration.

The arrangement of our lines and production equipment, the location of the control stations, and the facilities of our quality control laboratory allow us reliable mechanical and metallurgical control of the products in all phases of the process from raw materials to finished products.

We work with national and international specifications and standards demanded by our customers such as the following: ASTM, AREA, UIC, SCT, N. de M., FCP, CNCF, NOM, and those specifications that are eventually required by those countries to which we export.

The national and international acceptance of the quality of our products, the outcome of 16 years of experience, is the sequel of following the performance of our products used on railroad tracks and the facilities of our Development and Investigation Laboratory.

In this Laboratory we have electro-hydraulic equipment that permits simulation of work conditions and accomplishment of static and dynamic tests required by specifications and, something very important, development of new products.

I thank you for your kind attention and want to express our sincere wish that AREA's Regional Meeting results in complete success. At the same time we hope that your stay in our country will be very pleasant.

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