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Numerical Index to Committee Reports

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JANUARY 1993

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No. 739

JANUARY 1993

Proceedings Volume 94 (1993)

Stefanie Streever, Editor

CONTENTS

Front Cover: Union Pacific double stack train in Feather River Canyon near Keddie, California.

Rear Cover: Westbound Union Pacific double stack train crosses under itself west of Portola, California on Williams Loop, which allows a 1% maximum grade to be maintained in this mountainous territory.

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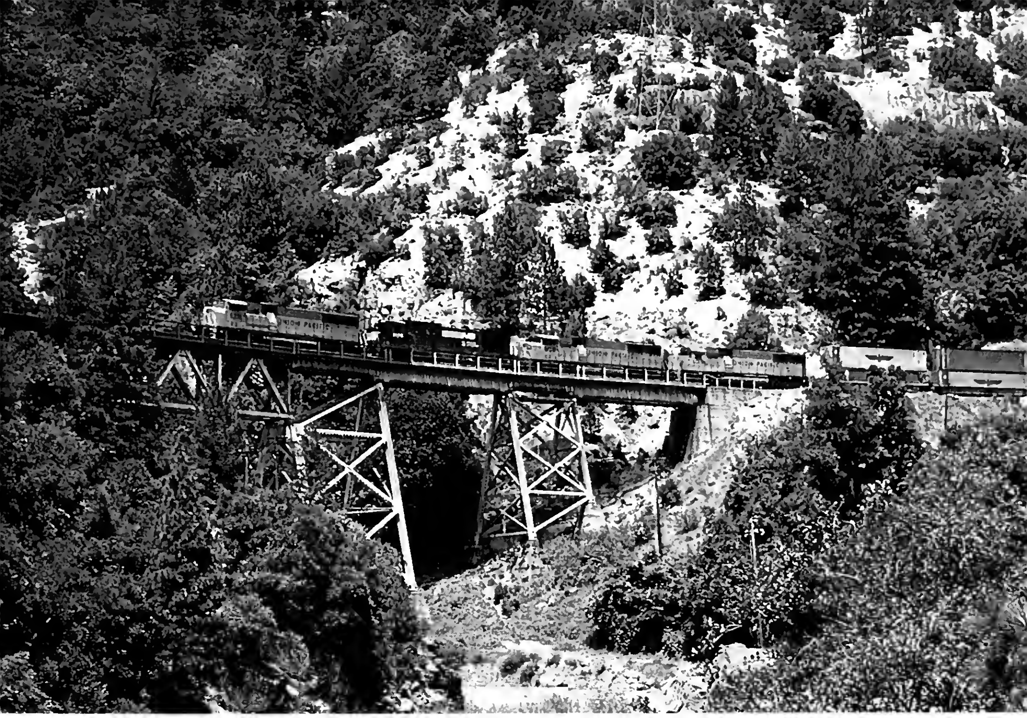

Union Pacific double-stack train crosses bridge on Salt Lake City-Oakland Line in California's Feather River Canyon.

Proposed Revisions to Chapter 15 - Steel Structures

The Manual revisions to Chapter 15 involve the deletion of the entire Part 2 and combining its provisions in Part 1 and revising the affected pages of other parts.

These extensive changes are not intended to affect substantively any of the specifications presently contained in Parts 1 and 2.

The following narrative is included to clarify and illustrate the needed revisions.

AREA Committee 15, Combining Parts 1 and 2 of Chapter 15, AREA Manual. Revisions as approved by 1991 Letter Ballot No. 3.

General Notes: This listing of revisions does not include redating affected pages, redating items in title pages and tables of contents, nor revising footnotes covering "page consists". Also excluded is the revision of the index at the beginning of the Manual.

Page x at the beginning of the AREA Manual: In the listing for Chapter 15 contents, replace "Part 2 - Design - High Strength Steels" with "Part 2 - Combined with Part 1 in 1993".

Chapter 15 (yellow) title page: In the "Notes on Key Features of the Manual," change the page numbering example from Page 15-2-1 to Page 15-3-4.

Page i: In the first line of the FOREWORD, insert "and 3" after "Part 1". Add a new third paragraph to the FOREWORD reading "Former Part 2 - Design - High Strength Steels was deleted in 1993, when its provisions were incorporated in Part 1". Change the first line of THE TABLE OF CONTENTS to read "Part 1 - Design (1993)".

Page ii: Confine Part 2 listing to "Part 2 - Design - High Strength Steels" (Combined with Part 1 in 1993)

Page iv: Delete the reference to Section 9.2.

Page 15-1-1: Near the top of page, delete "-ASTM A36 Steel," retaining the superscript 1. In the FOREWORD delete all words after "structural steel". Note that all of the references in Footnote 1 on Page 15-2-1 are already included among the references in Footnote 1 on Page 15-1-1, making it unnecessary to revise the latter footnote.

Page 15-1-3: In the listing of structural steel in Art. 1.2.1, delete the reference to A36 and A709 and substitute "Table 1.2.1A". In the text below the listing of structural steel change the table reference for impact testing from 1.2.1 to 1.2.1B. For rivet steel, revise the reference to read "A502, Grade 1 or Grade 2". In the footnote delete the reference to Art. 9.2.2.1.

Page 15-1-4: For "Cast steel, for shoes" list "A27 or A148". In the listing for "Forged steel..". delete "Class D".

Page 15-1-4: New TABLES 1.2.1A and 1.2.1B, as shown on Pages 8 and 9, will combine present tables 1.2.1, 2.2.1A and 2.2.1B, without reference to "HIGH STRENGTH" in the titles, and with notes brought up to date.

Page 15-1-13: In Art. 1.3.13, delete the reference to Section 2.4.

Page 15-1-17: Near the middle of Art. 1.3.14, replace 20,000 with ".55 F_y". Add to the article the definition "F_y = the yield point of the material as specified in Table 1.2.1A". In Art. 1.3.14.2, substitute ".55 F_y (for value of F_y see Table 1.2.1A)" for "20,000 psi".

Pages 15-1-18 thru 15-1-21: Use the entire content of Art. 2.4(a) to replace Art. 1.4(a), changing the reference "Art. 2.2.1" to "Table 1.2.1A". Use the entire content of present Art. 2.4.1 to replace the content of present Art. 1.4.1, with these revisions to the content of present Art. 2.4.1: Delete "High Strength" from the article heading. In the listing for axial tension, delete "high strength". Under the listing for tension in floorbeam hangers, change "20,000 to '19800'". Under the listings for axial compression in members centrally loaded, rearrange the two line text for the middle range of kl/r (starting with "when 3388/...) to be on one line. In the listing for compression in extreme fibers of welded built-up or rolled-beam flexural members...., move the asterisk now applying to the second formula to the first formula, namely:

$$0.55F_y - \frac{0.55F^2{}_y(1/r_y)^{2*}}{1.8 \times 10^9}$$

In connection with compression in extreme fibers of box type welded, riveted or bolted built-up flexural members etc., change the article references to 1.6.1 and 1.6.2. Adjust the margins of text beginning with "Compression in extreme fibers of box type welded....," and move the immediately following formula to the right edge of page. In connection with bearing on pins, change the reference "Art. 2.2.1" to "Table 1.2.1A". In connection with bearing on A325 and A490 bolts and the accompanying definition of F_u, delete the portion of the definition after "connected part, ksi" and substitute "as specified in Table 1.2.1A". In the double asterisk note in connection with bearing on A325 and A490 bolts, insert "F_p" after "bearing pressure," and change "is" in the last line to "are". In connection with bearing between rockers and rocker pins, change the reference "Art. 2.2.1" to "Table 1.2.1A". In connection with bearing on expansion rollers and rockers, change the reference "Art. 2.2.1" to "Table 1.2.1A" and delete "psi" from the definition of F_y. In the formula and accompanying definitions for bolts subjected to combined tension and shear, replace F_y with F_v. In the definition of T_b, delete "of Part 1".

Note that TABLE 1.4.1 is to remain unchanged.

Pages 15-1-21, 15-1-22: Use the entire content of present Art. 2.4.2 to replace the content of present Art. 1.4.2, with these revisions to the content of present Art. 2.4.2: In the first line change the reference "Art. 2.2.1" to "Table 1.2.1A". In the electrode listing under fillet welds, add "60,000 psi tensile strength.....16,500*," and for the 19,000 and 22,000 allowable stresses now listed, move the asterisks to the right side of the listing.

Page 15-1-22: For the present Art. 1.4.3, substitute Art. 2.4.3, changing the Art. No. to 1.4.3.

Pages 15-1-25, 15-1-26: Use the entire content of present Art. 2.6.1 to replace present Art. 1.6.1(a) and 1.6.1(b), making these revisions to the content of present Art. 2.6.1: In the definitions change the references from "Art. 2.4.1" to "Art. 1.4.1" and (in two places) from "Art. 2.2.1" to "Table 1.2.1A". Paragraph (c) of present Art. 2.6.1 is to be retained.

Page 15-1-26: Use the entire content of present Art. 2.6.2 to replace present Art. 1.6.2, making this revision to the content of present Art. 2.6.2: Under the definitions, change the article reference 2.2.1 to 1.2.1.

Page 15-1-27: Use the entire content of present Art. 2.6.3.1 to replace the content of present Art. 1.6.4.1, making this revision to the content of present Art. 2.6.3.1: Change the reference "Art. 2.2.1" to "Table 1.2.1A".

Page 15-1-28: In Art. 1.6.4.3(d), substitute "$\sqrt{F_y}/2300$ times" for "1/12 of". At the end of the affected sentence, add ", where F_y=the yield point as specified in Table 1.2.1A for the material, nor less than that specified in Art. 1.6.1(b) for solid plates".

Page 15-1-29: At the end of Art. 1.7.1(b), delete "157" and substitute, "$29{,}000/\sqrt{F_y}$, where F_y is the yield point as specified in table 1.2.1A for the material".

Page 15-1-29: Use the entire content of present Art. 2.7.2 to replace Art. 1.7.3, making these revisions to the content of present Art. 2.7.2: Change the first line to read "The thickness of webs of plate girders shall not be less than 1/6 the thickness of the flange nor less than." Substitute reference "Art. 1.4.1" for "Art. 2.4.1" and "Table 1.2.1A" for "Art. 2.2.1." Delete "psi" from the first and third definitions.

Page 15-1-31: Use the entire content of present Art. 2.7.3 to replace present Art. 1.7.8, making these revisions to the content of present Art. 2.7.3: Change the reference "Art. 2.2.1" to "Table 1.2.1A". In the definition of F_y delete "psi".

Page 15-1-34: In Art. 1.10.6, revise the first line of (b) to read "When a steel listed in AREA Table 1.2.1A is to be supplied....".

Page 15-1-40: Revise Footnote 5 under Table 1.14.7, to read "The suffix F is an ASTM A709 designation for fracture critical material requiring impact testing, with Supplemental Requirement S84 applying. A numeral 1, 2 or 3 shall be added to the F marking to indicate the applicable service temperature zone." Change the format of the thickness column of Table 1.14.7 as shown below.

Revision of thickness column of Table 1.14.7:

Thickness Inches
To 1-1/2 incl. Over 1-1/2 to 4 incl.
To 1-1/2 incl. bolted or riveted
Over 1-1/2 to 4 incl. bolted or riveted
To 1-1/2 incl. welded
Over 1-1/2 to 2 incl. welded
Over 2 to 4 incl. welded
To 1-1/4 incl. bolted or riveted

Pages 15-2-1 thru 15-2-11: Delete these pages.

Pages 15-4-1, 15-5-1 and 15-6-1: In the FOREWORD on each of these pages, delete the reference to Part 2.

Page 15-6-8: At the end of Art. 6.2.1(a), delete the reference to Art. 2.2.1.

Page 15-6-11: In Item 9, near the end of Art. 6.2.11, delete the reference to Part 2.

Page 15-6-19: In Art. 6.4.1, delete the reference to Part 2.

Page 15-7-1: In the FOREWORD, delete the reference to Part 2.

Page 15-7-3: In Art. 7.2.1.2(a), delete the reference to Part 2.

Page 15-7-4: Delete the comma in the last line of Art. 7.2.1.4(c). In Art. 7.2.1.4(d), delete the reference to Part 2.

Page 15-7-6: In Art. 7.2.3.1(a), delete the reference to Part 2.

Page 15-7-11: In Arts. 7.3.4.2, delete the reference to Art. 2.3.1 in paragraphs (a), (b) and (e).

Page 15-7-13: In the listing for compression in extreme fibers of box type welded.... members, etc., change the article references 2.6.1 and 2.6.2 to 1.6.1 and 1.6.2.

Page 15-8-1: In the FOREWORD, delete the reference to Part 2.

Page 15-8-3: In Art. 8.1.3.1(a), change the part referencing, beginning at the end of the first line, to "Parts 1 and 3 through 6".

Pages 15-8-15 and 15-8-16: In Arts. 8.4.2(a), 8.4.3(b) and 8.4.7(a), change the part referencing to "Parts 1 and 3 through 5 of Chapter 15".

Page 15-9-1: In the FOREWORD, change the end of the first line to read "Parts 1 and 3 through 8". In the CONTENTS, delete "and 9.2".

Page 15-9-2: In the top line of this page, delete "and 9.2". List Art. 9.1.2.1 only, deleting "and 9.2.2.1". Change the second paragraph of Art. 9.1.2.1 to read "Tables 1.2.1B and 1.14.7 make provisions for....". Revise the first sentence of the third paragraph of Art. 9.1.2.1 to read "Refer to Tables 1.2.1B and 1.14.7." Delete the second sentence of the third paragraph of Art. 9.1.2.1. In the fifth paragraph of Art. 9.1.2.1, change three table references from 2.2.1B to 1.2.1B. In the last sentence of Article 9.1.2.1, delete the reference to Art. 9.2.3.1.

Page 15-9-8: List Art. 9.1.3.13 only, deleting "and 9.2.3.1". Revise the first line of the third paragraph of the article to read "The fatigue formulas in Parts 1 and 2 of the 1969 edition of these specifications, now combined as Part 1, were based on....".

Page 15-9-13: List Art. 9.1.3.14.1 only, deleting "and 9.2.3.2.1". In the first line following the formulas, delete the reference to Art. 2.4.1.

Page 15-9-14: List Art. 9.1.3.16 only, deleting "and 9.2.3.3". Delete the first paragraph of Art. 9.1.3.15 and "also" from the first line of the second paragraph.

Page 15-9-15: List Art. 9.1.4 only, deleting "and 9.2.4". Delete the first sentence of Art.9.1.4.

Page 15-9-16: In the text near the bottom of this page, there are references to Arts. 2.4.1(a), and 2.7.1(b) and 2.4.1(a). These references should be changed to Arts. 1.4.1, 1.7.1(b) and 1.4.1, respectively.

Page 15-9-17: There are five references on this page to Art. 2.4.1(a). Change these to Art. 1.4.1.

Page 15-9-18: List Art. 9.1.4.2 only, deleting "and 9.2.4.2". In the text of Art. 9.1.4.2, delete the reference to Art. 2.4.2(a). Art. 9.1.4.3 only should be listed, deleting "and 9.2.4.3".

Page 15-9-20: List Art. 9.1.6.1 only, deleting "and 9.2.6.1". In the second line of Art. 9.1.6.1 text, change the article reference to 1.6.1(b). Below the second formula there are two sentences in parentheses, both of which are to be deleted. In the last two lines on this page, delete the references to Arts. 2.6.1(c) and 9.2.6.3.

Page 15-9-21: List Art. 9.1.6.2 only, deleting "and 9.2.6.2". Revise the beginning of the last paragraph of Art. 9.1.6.2 to read "In determining the values specified in Art. 1.6.2, conservative modifications....". List Art. 9.1.6.4 only, deleting "and 9.2.6.3".

Page 15-9-22: List Art. 9.1.7.1 only, deleting "and 9.2.7.1". In the third paragraph of Art. 9.1.7.1 delete "= 157 where F_y = 36,000 psi)". In the next to last line of Art. 9.1.7.1, delete the reference to Art. 9.2.4". List Art. 9.1.7.3 only, deleting "and 9.2.7.2". Delete the last sentence of the first paragraph of Art. 9.1.7.3.

Page 15-9-23: List Art. 9.1.7.8 only, deleting "and 9.2.7.3". In the third line of Art. 9.1.7.8 text, change the article reference to 1.7.8.

Page 15-9-26: At the beginning of Art. 9.1.14.7 add a paragraph reading "For comments relating to Table 1.14.7, see Art. 9.1.2.1."

Page 15-9-33: In the third paragraph of Art. 9.7.3.4.2, delete the reference to Art. 2.3.1.

Pages 15-9-35 and 15-9-36: Delete the article references listed in the following tabulation:

Subject	Article References to be deleted
Allowable stresses - base metal	2.3.1; 2.4.1
Allowable stresses - weld metal	2.4.1(a); 2.4.2; 9.2.4
Bridge welding code, AWS D1.5	9.2.4.2
Compression in welded box-type flexural members	2.4.1(a)
Compression in welded built-up flexural members	2.4.1(a); 2.4.1(e); 2.6.1(b); 2.6.2(b); 9.2.7.1
Fatigue	2.3.1
Plate girder	2.7.1; 2.7.3(a); 9.2.7.1
Stiffener plate	2.7.3(a)(b)
Welding electrodes	2.2.1

Insert the following tables 1.2.1A and 1.2.1B in or near Art. 1.2.1:

Table 1.2.1A — STRUCTURAL STEEL

ASTM Designation	F_y = Min Yield Point psi[1]	F_u = Min Ultimate Strength psi[1]	Thickness Limitation: For Plates and Bars inches[1]	Thickness Limitation: Applicable to Shapes of Groups[1,2]
A36[5]	36,000	58,000	To 6 incl	All
A709, Grade 36	36,000	58,000	To 4 incl	All
A588[4] A709, Grade 50W[4]	50,000	70,000	To 4 incl	All
A588[4]	46,000	67,000	Over 4 to 5 incl	None
A588[4]	42,000	63,000	Over 5 to 8 incl	None

Table continued next page.

Table 1.2.1A — STRUCTURAL STEEL (Continued)

ASTM Designation	F_y = Min Yield Point psi[1]	F_u = Min Ultimate Strength psi[1]	Thickness Limitation: For Plates and Bars inches[1]	Thickness Limitation: Applicable to Shapes of Groups[1,2]
A572, Grade 60[3,5]	60,000	75,000	To 1-1/4 incl	Groups 1 and 2
A572, Grade 50[5] A709, Grade 50	50,000	65,000	To 4 incl	All
A572, Grade 42[5]	42,000	60,000	To 6 incl	All

[1]These data are for information only and are current as of January 1991.

[2]Groups 1 and 2 include: W shapes 40 x 149 to 268 incl; 36 x 135 to 210 incl; 33 x 118 to 152 incl; 30 x 90 to 211 incl; 27 x 84 to 178 incl; 24 x 55 to 162 incl; 21 x 44 to 147 incl; 18 x 35 to 143 incl; 16 x 26 to 100 incl; 14 x 22 to 132 incl; 12 x 14 to 106 incl; 10 x 12 to 112 incl; 8 x 10 to 67 incl; 6 x 9 to 25 incl; 5 x 16 & 19; 4 x 13. M shapes to 37.7 lb/ft. All S, C and MC shapes. HP shapes to 102 lb/ft. All L shapes to 3/4 in. thickness incl. For a complete list of size groups see ASTM A6.

[3]A572, Grade 60, is to be used only for riveted or bolted construction.

[4]A588 and A709, Grade 50W have atmospheric corrosion resistance of approximately 4 times that of A36 steel without copper.

[5]For welded bridge construction, the material shall not be rimmed or capped steel.

Table 1.2.1B — IMPACT TEST REQUIREMENTS[1] FOR STRUCTURAL STEEL — OTHER THAN FRACTURE-CRITICAL MEMBERS[4]

ASTM Designation	Thickness Inches	Minimum Average Energy, Ft-lb and Test Temperatures		
A36	To 6 incl	15@ 70°F	15@ 40°F	15 @10°F
A709, Grade 36T[5]	To 4 incl	15@ 70°F	15@ 40°F	15@ 10°F
A588[2] A572, Grade 50[2] A709, Grade 50T[2,5] A709, Grade 50WT[2,5]	To 4 incl bolted or riveted	15@ 70°F	15@ 40°F	15@ 10°F
	To 2 incl welded	15@ 70°F	15@ 40°F	15@ 10°F
	Over 2 to 4 incl welded	20@ 70°F	20@ 40°F	20@ 10°F
Minimum Service Temperature[3]		0°F	-30°F	-60°F

[1]Impact tests shall be in accordance with the Charpy V-Notch (CVN) tests as governed by ASTM Specification A673 with frequency of testing H.

[2]If the yield point of the material exceeds 65 ksi, the test temperature for the minimum average energy required shall be reduced 15 ° F for each increment or fraction of 10 ksi above 65 ksi.

[3]Minimum service temperature of 0°F corresponds to Zone 1, -30 ° F to Zone 2, -60 ° F to Zone 3, referred to in Article 9.1.2.1.

[4]Impact test requirements for structural steel of Fracture Critical Members are specified in Table 1.14.7.

[5]The suffix T is an ASTM A709 designation for non-fracture critical material requiring impact testing, with Supplemental Requirement S83 applying. A numeral 1, 2 or 3 shall be added to the T marking to indicate the applicable service temperature zone.

Excerpts From Annual Report of A.R.E.A. Committee 1 — Roadway and Ballast

W. L. Heide, Chairman

Brief Status of Each Subcommittee Assignment:

Subcommittee A. Recommendations for Further Study and Research

No recommendations from this subcommittee at this time. Continue this subcommittee for future study and research recommendations.

Subcommittee B. Revision of Manual

Manual updates are being progressed with enthusiasm on most subcommittees.

Subcommittee 1. Roadbed

D1-2-86. Assignment is to update the Manual Sections 1.1 through 1.4 on the exploration and testing, design, construction and maintenance of roadbed. Assignment is on-going. A completion in the Fall of 1993 is expected.

Subcommittee 2. Ballast

C2-1-86. A continuing assignment to provide monitoring and recommendations regarding the AAR ballast research program and other test installations. The subcommittee is monitoring the FAST/HAL ballast and subgrade tests.

D2-1-88. Assignment is to investigate and report ballast tamping methods and procedures and their relations to ballast section compaction. This assignment has progressed slowly and is still in the planning stage to do the field study. Assignment will be reviewed in 1993 for continuation plans.

D2-2-89. Assignment is on the study and report of performance characteristics of hot plant mixed asphalt courses in the track substructure. Various field projects have used HMA and are being analyzed for performance and economics. Report is expected by Fall, 1993.

D2-3-89. Assignment is to study the relationship between in-track performance of ballast materials and laboratory results obtained by the Mill Abrasion test. Lab tests have been done and are being analyzed. A report is expected by Fall, 1993.

Subcommittee 3. Natural Waterways

No new assignments.

Subcommittee 4. Culverts

D4-1-88 Assignment is to update the Manual text on installation of pipe culverts. This assignment is on-going and expected to be completed by Fall, 1993.

D4-2-92. Assignment is to develop Manual recommendations for procedures for culvert inspection. This assignment has recently started, is on-going and expected to be completed by Fall, 1994.

Subcommittee 5. Pipelines

D5-2-81. Assignment is to study the use of plastic carrier pipes. Some progress has been made and it is on-going with expected report by Spring, 1994.

D5-3-91. Assignment is to develop specifications for uncased high pressure gas pipelines within the railway right-of-way which has been progressed to completion and letter ballot approved. A tremendous effort was put forth by the subcommittee to complete this assignment.

D5-4-92. Assignment is to prepare specifications for overhead pipeline crossings and will be an on-going assignment to be completed by Spring, 1994.

D5-5-92. Assignment was to study and report on directional boring of pipe under track. A brief report was completed and published in the December Bulletin as information.

Subcommittee 6. Fences.

D6-1-87. Assignment dealing with security fencing is proceeding slowly and could use some expertise to help in providing recommendations for security fencing for Manual material.

D6-2-87. Assignment on the review and updating of Manual recommendations for snow fencing has progressed well. Proposed recommendations will be reviewed in 1993 for completion of this assignment.

Subcommittee 7. Signs

D7-1-91. Assignment on the review of Part 7 in the Manual is on-going and completion is expected in 1993.

Subcommittee 8. Tunnels.

D8-1-87. Assignment to review and update Manual recommendations for tunnels is going slowly and should progress some during 1993 with completion in 1994.

D8-2-87. Assignment to report on the Rogers Pass Project on the Canadian Pacific is still on-going with completion expected by Fall, 1993.

Subcommittee 9. Vegetation Control

D9-2-89. Assignment to review and update Manual text recommendations for vegetation control is on-going and is expected to be complete by Fall, 1993.

Subcommittee 10. Geosynthetics

D10-3-88. Assignment has progressed to a completed and approved report on the use of geosynthetics in retaining walls and slope stabilization. The report will be published in the May Bulletin as information.

D10-1-89. Assignment to develop recommended practices for the use of geosynthetics for stabilization of the track roadbed is on-going. A draft is expected by Fall, 1993.

The subcommittee is in the process of sending out a questionnaire to various geotextile suppliers on test results for abrasion resistance in order to set forth Abrasion Test parameters to complete Table 10, 1.2.2 in the Manual.

Excerpts From Annual Report of A.R.E.A. Committee 2 - Track Measuring Systems

R. F. Silbaugh, Chairman

Status of Subcommittee Assignments:

Subcommittee A — Recommendations for further study and research.

D1-1-93. Survey of track measuring vehicles.

This new task will cover geometry, detector, corrugation, clearance, railhead profile and other measurement vehicles in North America.

D2-1-93. Develop guidelines for the use of automated rail measurement in the planning of rail grinding and rail renewal.

Subcommittee A — Recommendations for assignments to be dropped.

C1-1-85. Survey data processing techniques and equipment used on different railway track geometry vehicles.

Proposed new task D1-1-93 will be used to update and expand scope of this task.

C1-1-87. Proposed recommended practice for geometry cars. (Incorporated into current assignment D1-2-92)

Subcommittee B — Revision of Manual

The following new articles have been published in the 1992-1993 AREA Manual.

1.3 Compilation of Various Track Geometry Parameters
2.1 Description of a Generic Track Geometry Measuring Vehicle
2.2 Recommended Minimum Performance Guidelines for Rail Testing

COMMITTEE 2 — REPORT ON SPECIAL (D) AND CONTINUING (C) ASSIGNMENTS

1. Assigned Number	**D1-1-92**	**D1-2-92**	**D2-1-92**	**D3-1-92**	**D3-2-92**	**C1-1-87**	**C1-1-85**
2. Description of Assignment	Measure Techniques	Locate Defects	Measure Rail Parameters	Gage Strength Measurement	Track Lateral Stability	Recommended Practice Geom. Car	Survey Processing Techn. & Equip.
3. Date Assigned/ (Undertaken)	12-5-91	12-5-91	12-5-91	12-5-91	12-5-91	3-1-87	3-1-85
4. Estimated % Completion	90%	10%	90%	50%	10%		
5. Estimated Completion Date	3/93	7/94	3/93	9/93	4/94		
6. Problems	None	None	None	None	None		
7. Recommend Assignment Being Dropped/ Discontinued	No	No	No	No	No	Drop*	Drop**
8. Recommend Assignment Being Postponed	No	No	No	No	No		

*Incorporated into D1-2-92 **Proposed new Task D1-1-93 will be used to update & expand scope.

Excerpts From Annual Report of A.R.E.A. Committee 3 - Ties and Wood Preservation

J. L. Watt, Chairman

Status of Subcommitee Assignments:

Subcommittee A — Recommendation for further study and research.

Subcommittee B — Revision of manual.

Both of these subcommittee activities are incorporated into Subcommittees 1-4 and are not currently chaired.

Subcommittee 1 — Cross and Switch Ties

C1-2-63. Keep up-to-date specifications for cross and switch ties.

A.R.E.A. and RTA specifications were combined in 1988. A new industrial grade crosstie specification has been submitted to the A.R.E.A. Board on October 9, 1992 for approval.

C1-1-62. Investigate possible revision of crosstie design and/or spacing.

AAR has just published data which suggests that larger ties on wider spacings last longer. No economic justification has been proven though.

D1-1-89. Wood tie disposal alternatives.

Questionnaires from railroads and tie producers are being evaluated for feasible tie disposal options.

Subcommittee 2 — Wood preservatives and preservative treatment of forest products.

C2-1-63. Keep up-to-date specifications for preservatives and update research for new preservatives.

The proposed AWPA changes in Chapter 6 were recently voted and approved by T-6 after further clarification and review of the data. A.R.E.A. has sent several position statements and recommendations on certain changes which will strengthen the requirements for proper treatment.

C2-2-63. Keep up-to-date specifications for seasoning, conditioning, and treatment.

A.R.E.A. is recommending the use of environmentally safer techniques which lessen the liability of railroads that use treated material along their properties. Stripping each layer of air-dried crossties for treatment and using vacuum pumps instead of steam jets lessens the preservative which can bleed from ties after shipment.

C2-3-63. Advisability of preparing specifications to cover care and handling of forest products before and after treatment.

A.R.E.A. recommends use of reusable stickers so that our right-of-ways aren't littered with broken wooden sticks.

Subcommittee 3 — Service records of forest products.

C3-1-63. Inform A.R.E.A. about annual tie renewal statistics as furnished by the Economics and Finance Department - AAR.

Information concerning annual tie renewal is furnished each year.

C3-2-63. Keep current with service test records of forest products used in railroad construction and maintenance.

The AAR computer list is updated and test sites visited periodically on field trips. The Des Plaines, Illinois site for laminated ties with different spacings was visited June 15, 1992.

Subcommittee 4 — Collaborate with AAR Research Department and other organizations in research and other matters of mutual interest.

C4-1-76. Subcommittee for solid-sawn ties.

Glue-lam and dowel-laminated ties continue to perform well in tests. CP Rail has some over 40 years old which we visited in June, 1991. The Cedrite plant is not in production now. Recent NS installations did not perform well in warm, wet locations. As noted above, the Des Plaines, Illinois site proves that laminated ties have definite future potential.

C4-3-63. Splitting of ties and anti-splitting devices.

Nail plates have proven to be effective split control devices.

C4-3-66. Wood deterioration in the presence of metal.

Wood deterioration from decay and metal sickness occur simultaneously. AAR has devised a test to partially diagnose how much deterioration is caused by metal present in the wood, but it's difficult to put numbers on the two modes.

C4-4-82. Monitor progress of subcommittee work and assist with an appropriate technology exchange between AAR and A.R.E.A.

Good data exchange occurs now. Several projects are now under way which involve the AAR, A.R.E.A., and RTA. The test inventory data compiled on an AAR computer is one example of this.

Excerpts From Annual Report of A.R.E.A. Committee 4 - Rail

H. W. Newell III, Chairman

Report on Special (D) and Continuing (C) Assignments

1. Assigned Number	**C1-1-74**	**D1-1-81**	**C2-1-55**	**D2-1-88**	**D2-2-88**	**C3-1-76**	**C4-1-73**	**C5-1-78**
2. Description of Assignment	Collaborate w/Rail Welding, Grinding & Test. Cont.	High Strength Rail Welding	Collaborate w/Rail & Joint Bar Manufacturers	Web Cracking of Rail from Roll Straightening	Tolerance and Method of Meas. Rail Head Radius	Rail Statistics	Update Data on Methods & Equip. for Weld Repair	Rail Specs. Research & Develpmt.
3. Date Assigned/ (Undertaken)	1974	1981 (or earlier)	1955	1988	1988	1976	1973	1978
4. Estimated % Completion	Continuing	80%	Continuing	98%	75%	Continuing	Continuing	Continuing
5. Estimated Completion Date	N/A	1993	N/A	1993	1993	Annual	N/A	N/A
6. Problems	NIL	Redrafting	NIL	Certification Process to be determined	Included in gages	NIL	NIL	NIL
7. Recommend Assignment Being Dropped/ Discontinued	Yes, Assocs., now attend. No need for this subcommittee	No	Yes, Assocs. now attend. No need for subcommittee	No	No	No	No	No
8. Recommend Assignment Being Postponed	No	No	No	No	No	No	No	No

A.R.E.A. Committee 4 - Rail (cont.)

Report on Special (D) and Continuing (C) Assignments (Con't)

1. Assigned Number	**D5-2-88**	**C5-1-78**	**C6-1-62**	**C7-1-68**	**C8-1-88**	**D5-1-92**	**D5-2-92**	**D6-1-92**
2. Description of Assignment	Wheel/Rail Profiles	Standard Mill Check Gauges for Rail	Joint Bars; Specifications Testing	Effect of Heavy Wheel Loads on Rail	Management of Rail Test Resources	Standardize RR Specs.	Hydrogen Level & Tensile Bar Requirements	Specifications for Encapsulated Joint Bars & Fiberglass Joint Bars
3. Date Assigned/ (Undertaken)	1988	1989	1962	1968	1988	1992	1992	1992
4. Estimated % Completion	50%	75%	Continuing	Continuing	100% Complete	5%	20%	20%
5. Estimated Completion Date	1993+	1993	N/A	N/A	1993	1995	1994	1994
6. Problems	Very complex subj. Optimum head contour function of track curvature	NIL	NIL	None	Need to re-ballot insufficient response	None	None	None
7. Recommend Assignment Being Dropped/ Discontinued	No	No	No	No	Yes	No	No	No
8. Recommend Assignment Being Postponed	No	No	No	No	No	No	No	No

Excerpts From Annual Report of A.R.E.A. Committee 5 - Track

J. D. Baker, Chairman

Report on Special (D) and Continuing (C) Assignments.

1. Assigned Number	**B-1-89**	**C2-1-79**	**D4-1-84**	**D4-1-92**	**D4-2-92**	**C5-1-90**
2. Description of Assignments	Swt. Pt. Chg. out	Track Tools	Rail Fast. Wood Ties	Elast. Fast. Spcl. Trkwk.	Trk. Evalu. Methodology	Update Trkwork plans
3. Assigned Date	12/89	2/79	4/86	10/92	10/92	12/90
4. Estimated Comp. percent (%)	0%	Continue	95%	0%	0%	Continue
5. Est. Compl. Date	10/92	Continue	5/91	12/95	12/95	Continue
6. Problems	None	None	None	None	None	None
7. Recomm. Assign. (Dropped/Disc.)	No	No	No	No	No	No
8. Recomm. Assign. (Postpn./Change)	No	No	No	1/93	1/93	No
Chairmans' Comments			AREA/AAR Testing			

1. Assigned Number	**D5-4-81**	**D5-8-89**	**D5-2-91**	**D8-2-84**	**D8-3-85**	**D8-4-89**
2. Descriptions of Assignments	Explosive Hard. Cast.	TTC T.O. Design	Fastn. for Frog & Cast	Supr. Elev. & Sprl. Lg.	T.O. Geomty. Incl. Speed	Vert. Cvr. Rate/Chg.
3. Assigned Date	1/81	5/89	3/91	10/89	10/89	10/89
4. Estimated Comp. percent (%)	95%	0%	0%	10%	10%	0%
5. Est. Compl. Date	10/91	10/92	12/93	5/93	5/93	5/94
6. Problems	None	None	None	None	None	None
7. Recomm. Assign. (Dropped/Disc.)	No	No	No	No	No	No
8. Recomm. Assign. (Postpn./Change)	No	No	No	No	No	No
Chairmans' Comments		AAR/TTC Evaluate	SEc. Lock Evaluate			

Excerpts From Annual Report of A.R.E.A. Committee 6 - Buildings and Support Facilities

J. H. Smith, Chairman

Brief status of each subcommittee assignment:

Subcommittee A. Recommendations for Further Study and Research

The committee is continuing to review subjects with a view towards identifying new subjects for manual information and material. This committee continually looks to future needs of the railroad industry. Approved for manual information has been a report by Mr. J. H. Super titled, "Interior Paint Systems and Application Methods." Other subjects under consideration are Impact on Railroad Buildings by the American Disabilities Act of 1990, Hardware and Keying in Buildings, Use of Personal Computer Based Building Management Systems, Management of Building Construction Contracts, Metrication in Railroad Building Design, and Building Energy Efficiencies.

Subcommittee B. Revision of Manual

This committee continues to review each committee chapter with a view towards upgrading material and contents.

C-1-73. Design Criteria for Spot Car Repair Shops

Submitted material for Committee Ballot of Approval on September 18, 1992. Unanimous ballot approval has been subsequently received from the committee membership.

D-3-87. Design Criteria for Wheel and Bearing Shops

Received approval from the committee on September 23, 1992 to submit material for Committee Ballot of Approval.

D-5-89. Design Criteria for Maintenance of Way Repair Facilities

Received approval from the committee on September 23, 1992 to submit material for Committee Ballot of Approval.

D-4-89. Design Criteria for Railway Material Management Facilities

Interim briefing advised on the progress of this report. All indications point to having this report ready for final review and ballot approval in 1993. This report was assigned in 1989. Railway Material Management Facilities continue to play a key role as part of any railroad company operations progressing towards the 21st century. This report will be completed in a timely manner.

Excerpts From Annual Report of A.R.E.A. Committee 7 - Timber Structures

J. A. Lileikis, Chairman

Subcommittee Status:

Assignment A. Recommendations for further study and research.

Committee 7 is investigating the possibility of establishing a subcommittee that deals with the disposal of timber bridge ties. Further topics for future subcommittee consideration are: Re-evaluation of allowable stresses currently used in the AREA Manual, Review techniques for inspection and evaluation of timber structure capacity, Establishing a history of past subcommittee assignments, and a thorough review of the Manual to determine which sections are still applicable.

Assignment B. Revision of Manual.

This subcommittee has just submitted to the membership recommendations for timber bridge tie specifications.

C-1-53. Specifications for Design of Wood Bridges and Trestles

At this time, typical timber structures have been analyzed for Cooper E-80 loading criteria. Theoretical load distributions on timber piles have been analyzed for 4, 5, 6, 7 and 8 pile bents.

C-2-90.Timber Technology Applications.

This subcommittee has been actively investigating laminated timber designs and their applications to the railroad industry. This subcommittee works closely with the University of West Virginia, and also the Forest Products Laboratory.

D-4-82. Effect of Unit Trains on Timber Trestle Components.

A new questionnaire has been sent out to the membership, and once results are compiled, the direction of the subcommittee will be determined. It is intended that this subcommittee will be able to analyze the hazardous effects of unit train traffic over timber structures.

D-5-89. Upgrading Existing Timber Bridges.

A final draft on recommended practices for moisture control on timber structures, application of preservative chemicals, and the support, repair, and/or replacement of decayed/damaged portions of timber structures is being prepared. It is anticipated that this subcommittee will be concluded with their work in 1993.

D-6-89. Specifications for Timber Bridge Ties.

A third draft copy of the specifications is required to be submitted to the membership due to the extensive revisions brought out by the first two drafts. It is anticipated that this subcommittee should be completed by the end of 1993, barring any further changes.

Excerpts From Annual Report of A.R.E.A. Committee 8 - Concrete Structures and Foundations

G. W. Cooke, Chairman

Subcommittee Activities:

Subcommittee A. Recommendations for Further Study and Research

We are participating in the AAR, NSF, and University of Illinois Research Program where they are investigating Dynamic Impact on Railroad Structures. An advance copy of the Final Report "Impact Factor Measurements for Three Precast Pretensioned Concrete Railway Bridges" is now being reviewed by the Bridge Research Committee.

Subcommittee B. Revision of the Manual

This committee reviews all Manual Material in order to verify that it is in keeping with the Standard AREA Format. This Committee consists of only one member since most of the subcommittee work consists of correcting the format for publication.

Subcommittee 1. Design of Concrete Structures

D1-1-84. Develop Specifications for Precast and Cast-In-Place Concrete Segmental Bridges.

This assignment is out for Letter Ballot, and has received the votes to pass. The subcommittee is working on the revisions which will probably not be completed in time for publication.

D1-1-90. Concrete Slab Track. Formulate a recommended practice for the design and construction of concrete slab track with a direct fixation guideway for the rail (collaborating with Committees 5,10,12).

This assignment is now underway with the specification 90% written. The preliminary draft is out for review of all concerned.

D1-5-87. Investigate applicable Impact Factors.

This assignment is dormant at the moment awaiting the results of the AAR Bridge Testing which is underway.

D1-9-88. Review and Rewrite Part 17 as Necessary.

This assignment is underway with the specification review started within the subcommittee. A draft specification is being circulated within the subcommittee. This assignment has suffered due to the large task of developing two other new specifications.

Subcommittee 2. Foundation and Earth Pressures

C2-2-90. Design of Proprietary Walls. Monitor Committee 1's Assignment D1-3-88 and provide a specification for design when and if necessary.

We are experiencing some problems in developing this specification. The other organizations which have developed specifications are constantly changing them. We are now getting the type of direction that we need, but the progress is slower than we would like.

D2-1-89. Review and Rewrite Part 20 as Necessary.

This assignment is complete having passed the ballot for Manual Material.

D2-2-89. Commentary for Part 20 on Flexible Sheet Pile Bulkheads.

A preliminary draft of the Commentary is under review by the subcommittee.

D2-2-90. Temporary Protection for Construction. Develop a specification for the design and execution of temporary shoring of the track structure.

This assignment is underway and represents new material for the Manual. Due to the nature of the material being produced, extensive research has been necessary, and extra planning by the members has resulted in slow progress.

Subcommittee 3. Durability of Concrete

C3-1-88. Monitor Current Waterproofing Practices.

This assignment is a continuing assignment, and no material has come out of the subcommittee this year.

C3-2-89. Monitor and Revise Part 1.

Specific activities have been undertaken as special assignments below.

C3-1-90. Prepare Commentary for Part 1.

This assignment is continuing and Manual Material has been approved and submitted for publication.

D3-2-90. Prepare Article on Concrete Durability.

This assignment is complete pending typing of the final draft. The general committee has approved the publication "For Information" in the Bulletin.

D3-3-90. Manual Material on Alkaline Reactivity.

Work is proceeding on this assignment which has been delayed in order to complete other material.

D3-4-90. Manual Material on Fiber Reinforced Concrete.

Work is proceeding on this assignment which has been delayed in order to complete other material.

D3-1-92. Manual Material on Sulfur Concrete.

Work is proceeding on this assignment.

Subcommittee 4. Repair, Restoration and Strengthening of Concrete Structures

D4-4-88. Develop Criteria for Prevention of Scour Damage to Bridge Piers.

The assignment is well underway and the subcommittee reports progress toward the goal of Manual Material.

D4-1-91. Develop a Commentary for Part 14.

Work is underway, and the Commentary is being expanded to add the new material from approval of the Part 14 Specification. Work on this assignment has been delayed due to the extensive work load of the subcommittee.

D4-1-92. Collision Protection Commentary.

The material has been approved by the subcommittee, and the general committee has approved the material for Letter Ballot.

Subcommittee 5. Design of Concrete Bridge Components in Seismic Zones

D5-1-89. Develop Design Criteria or Retrofit Specifications or Guidelines.

The subcommittee is awaiting progress with the formation of the New Committee for Seismic Design.

Subcommittee 6. Design of Foundations

D6-3-89. Commentary for Part 4 on Pile Foundations.

This assignment is 90% complete, and the subcommittee reports that a Letter Ballot will be coming next year.

D6-1-92. Addition to Part 3 Footing Foundations on Combined Footings.

This assignment is just underway, and the subcommittee reports initial progress.

D6-2-92. Commentary for Part 22 Geotechnical Subsurface Investigations.

This assignment is just underway, and the subcommittee reports initial progress.

Excerpts From Annual Report of A.R.E.A. Committee 9 - Highway-Railway Crossings*

K. R. Autenrieth, Chairman

Status of Subcommittee Assignments:

Subcommittee A. Recommendations for Further Study and Research

This subcommittee is responsible for the development of new topics which should be addressed in the manual. This subcommittee recently focused on the issue of clearances which graduated into a full subcommittee assignment (D-4-90). Recent brainstorming sessions with the full committee has given this subcommittee a full wish list of possible assignments to sift through. We are concentrating only on those which will lead to possible revision in the manual.

Subcommittee B. Revision of Manual

This subcommittee is responsible for the processing of manual revisions through the committee. A recent proposal on clearances for highway structures over railroads is now in final draft and will soon appear for formal Board and Committee approval. This subcommittee is discussing the possibility of language on grade crossing closures.

D-1-87. Foundations for Highway-Railway Grade Crossings

Current focus is on work necessary beneath the crossing surface when installing the various types of crossing surfaces. This subcommittee is exploring the use of asphalt underlayment and working toward developing specifications for inclusion in Chapter 1 (Committee 1). Once developed, Chapter 9 would cross-reference Chapter 1 specifications. This subcommittee is also reviewing cross section diagrams for drain pipes and filter fabric for possible inclusion in the manual.

We expect to work with Subcommittee C-1-87 to investigate the distance on either side of the crossing where rail joints should be eliminated.

C-1-87. Grade Crossing Surfaces

Most of this subcommittee's time is spent on the subject of crossing surface materials. They are undecided on the content and form of what should appear in the manual. The current idea is to publish a catalog chart of what is available on the current market from the supply industry and publish it as bulletin information only. We basically would take over the current annual list put together and published annually in *Track & Structures*.

This subcommittee would like to look at several recent outside reports on fasteners for the different types of surfaces. We are also interested in the subject of expected life-cycle spans for different surfaces.

D-2-87. Approaches to Highway-Railway Grade Crossings

This subcommittee is monitoring research relative to required approach grades for low clearance, long wheel base vehicles. This includes humped and sagged crossings. We are awaiting results from the Michigan Department of Transportation on their testing of computer software developed by Dr. Ron Eck.

Existing manual language is being reviewed to distinguish between new construction and reconstruction of existing crossings.

D-3-87. Grade Crossing and Separation Elimination

This subcommittee has finalized their evaluation of a lengthy report by Ernst & Whinney on grade separation elimination. It is our intent to publish their findings as bulletin information rather than as a manual revision.

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*At the December 3, 1992 A.R.E.A. Board of Direction meeting, it was voted to disband Committee 9, with assignments and Manual material being distributed to Committees 1, 5 and 28.

The subcommittee will investigate and review the FRA's recent announcement of a goal to close 25% of the nation's grade crossings. This support could also lead to future bulletin information.

D-4-90. Clearances for Highway Structures Over Railroad

A revision to the manual has been drafted in final form and has presented its findings to Subcommittee B for further handling. This subcommittee will remain intact until the new section has been approved and placed in the manual.

Excerpts From Annual Report of A.R.E.A. Committee 10 - Concrete Ties

J. F. Scott, Chairman

Subcommittee A. Recommendations for Further Study and Research.

It has been proposed that durability testing be conducted on various North American concrete tie samples in order to determine most appropriate test procedures for future avoidance of the deterioration problem which plagued certain ties in Canada and the U.S.

Subcommittee B. Revision of Manual.

Letter ballots on five separate items were sent out in late summer and early fall, and it is not known at time of writing if results will be received in time to make the October 15 manual revision submission deadline. On the first two ballots,there was poor response, and fewer than 50% of eligible voters had cast their ballots. It was decided to phone some of the members in an effort to get their votes.

Chapter 10 has now been drafted with Metric units in brackets after English units, and it was recommended at the September 16 meeting that such should go into the Manual after careful editing.

Subcommittee 1. Flexural Strength.

C1-1-87. Monitor developments in prestressed and reinforced concrete technology which may affect concrete tie requirements.

D1-2-89. Investigate impact resistance and design requirements for concrete ties.

D1-3-87. Evaluate acceptance criteria for repeated load tests.

No changes were made in Chapter 10 flexural strength requirements in the past year, but the performances of concrete ties on the FAST/HAL test track were closely monitored. As the full story has not yet been written on 125 tons, it is recommended that the current assignments be continued.

Subcommittee 2. Switch, Bridge and Crossing Ties.

D2-1-89. Investigate requirements for concrete switch ties, bridge ties, and grade crossing ties.

A complete new Section 1.11, *Ties for Turnouts*, has recently been balloted for inclusion in the Manual. At the September 16 meeting, it was requested that additional work be done looking at:

a) means of connecting switch ties end-to-end such that no single tie would be over 16 or 17 ft. long.
b) means of connecting guard rails to concrete ties used on ballasted deck bridges.
c) design requirements for road crossing ties.

This is an active subcommittee, and assignment should remain unchanged.

Subcommittee 3. Fastenings.

D3-1-83. Revise current test requirements.

D3-2-83. Investigate the effect of axle loads and tie spacing on fastening requirements.

D3-3-86. Review and recommend revisions of the load magnitude specified for the fastening repeated-load test.

These assignments should continue to ensure ability to cope with 125 ton cars.

Subcommittee 4. Durability.

D4-1-89. Resistance of concrete ties to alkali aggregate reaction.

D4-2-89. Resistance of concrete ties to freezing and thawing.

D4-3-89. Resistance of concrete ties to rail seat abrasion.

Assignments 1 and 2 have been dealt with in a proposed new Section 1.2, *Material*, which is currently under ballot. These assignments should not be dropped, however, as current standards around the world are undergoing changes, and there are bound to be revisions required in Manual material. Assignment 3 should continue as there currently are a great number of items under test on rail seats on the FAST/HAL loop at Pueblo.

Subcommittee 5. Maintenance Requirements.

C5-1-87. Maintenance requirements of concrete ties, including pads and insulation.

The results from a previous questionnaire have not yet been used to write maintenance requirements. This assignment should continue until such is completed.

Subcommittee 6. Ballast Requirements.

C6-1-87. Collaborate with Committee 1 on concrete tie ballast requirements.

As soon as the ballast requirements for concrete ties are adequately covered in Chapter 1, the section pertaining to ballast requirements in Chapter 10 will be deleted. Until then, this assignment should continue.

Subcommittee 7. Transit Tie Requirments.

C7-1-92. Collaborate with Committee 12 on transit tie requirements.

This is a new assignment with intent of ensuring that Committees 10 and 12 do not work at cross purposes and do not duplicate efforts with respect to concrete ties. The collaboration is seen as beneficial to both committees and should continue.

Excerpts From Annual Report of A.R.E.A. Committee 11 - Engineering Records & Property Accounting

A. R. Ranuio, Chairman

The committee held it's September 1991 meeting in Bismarck, North Dakota. This location offered the opportunity to observe the PB119 tracklaying machine in operation. During the business portion of our meeting we discussed the AAR endorsement of five of our proposals to the ICC as follows:

1. Option to eliminate accounting for Interest During Construction account 76.
2. Option to eliminate record keeping by Valuation Sections.
3. Option to no longer maintain signed Completion Reports on all capital projects.
4. Substitute a database for the Record of Property Changes manual ledger.
5. Revise list of acceptable reportable units.

The AAR asked us to get feedback from the Cost & Economics group for the following two proposals:

1. Move computers from account 59 equipment accounts to account 46 road accounts.

2. Eliminate charging of portion of account 46 to equipment accounts.

The committee held it's March 1992 meeting in Chicago. We received information from the ICC on our proposals. We also received a study of salvage rates, lives and disbursement curves from the ICC.

SUBCOMMITTEE 1 - ACCOUNTING

Charter:

To collect information about accounting changes that affects the railroad industry on an on-going basis. This includes collecting information about depreciation methods and standards.

Current Projects:

1. Explore methods of performing depreciation studies.

2. Explore Fixed Asset software for Accounting purposes.

3. Obtain updates on changes to ICC regulations.

4. Provide industry changes to existing ICC regulations i.e.:
   a. Eliminate ICC Account 76, Interest During Construction.
   b. Keep records by other than Valuation Section.
   c. Abolish ICC requirement for signed Completion Report and Record of Property Changes forms.
   d. Reclassify assessments from ICC account 39 to account 2.
   e. Revise ICC requirements for reportable units.

Future Activity Areas:

1. Explore opportunities for development of existing Fixed Asset programs into relational data bases that feed CADD Maps. Data would link cost of investment with operating costs, tax consequences, and revenue so that detailed maps could visually describe return on investment.

2. Other areas for future activity include:

   a. Substitute computer data for signed paper records.
   b. Raise the minimum capital amount from $5,000.
   c. Develop clearer methods for selecting depreciation salvage rates, service lives, and mortality curves.
   d. Develop method to write off elusive assets such as personal computers, communications radios and furniture.

Priority:

Studies related to depreciation have the most impact on railroads today, and this topic is the most time consuming and important.

SUBCOMMITTEE 2 — OFFICE & DRAFTING PRACTICES

Charter:

To collect information about office and drafting practices in the railroad industry on an on-going basis.

Current Projects:

There are no projects currently in this subcommittee.

Future Activity Areas:

This subcommittee is inactive.

Priority:

None

SUBCOMMITTEE 3 — TAXES

Charter:

To collect information about State and Federal Tax practices and changes that affect the railroad industry on an on-going basis.

Current Projects:

This committee reports current changes on an on-going basis.

Future Activity Areas:

This subcommittee will continue to report ongoing changes.

Priority:

Not applicable.

SUBCOMMITTEE 4 — PLANNING, BUDGETING & CONTROLS

Charter:

To collect information and identify standards for industry budgeting techniques.

Current Projects:

This committee is newly formed and has not yet begun its task.

Future Activity Areas:

This subcommittee will identify various budgeting techniques used within the industry with special emphasis on available software and in house computer programs.

Priority:

Not applicable.

Excerpts From Annual Report of A.R.E.A. Committee 12 — Rail Transit

C. D. Wylder, Chairman

Status of Subcommittees:

C-1-87. Rail Corridor Evaluation

a) Assigned - 1987

b) Progress to Date - Published Part 2 of Chapter 12 in the AREA Manual.

c) Next activity - Expand information in the manual and clean-up existing information. Working on summary of transit definitions.

d) Benefits - Provide a baseline source of information on transit corridor planning.

e) Problem areas - none.

f) Recommend that this group continue their work as they have produced a useful section of the manual and have additional information to publish.

C-2-87. Special Track/Roadway Considerations

a) Assigned - 1987

b) Progress to Date - Published in the AREA Bulletin a chart showing the design characteristics of each Transit Agency. They have prepared rough draft of Part 4 of Chapter 12 of the AREA Manual.

c) Publication of Part 4 will be in 1993. This will be an ongoing assignment even after initial publication as this area will be continually updated and added to.

d) Benefits - Will provide a source of recommended practices for track and roadway design.

e) Problem areas - none.

f) Recommend that this group continue their work as they are very close to manual publication.

C-3-87. Special Bridge/Structural Considerations

a) Assigned - 1987

b) Progress to Date - Have a rough draft of the Maintenance Facility Section of the Part 5 Chapter 12 of the Manual.

c) This assignment will be ongoing, as even after publication of initial information, there will be a need to add and update information.

d) Benefits - Will provide a source of recommended practices for bridges and structures.

e) Problem areas - none.

f) Recommend that this group continue their work as they are collecting valuable information.

C-4-88. Electrification

a) Assigned - 1988

b) Progress to Date - Have outlined information to go into Manual Chapter.

c) This assignment will be ongoing as even after publication of initial information, there will be a need to add and update information.

d) Benefits - Will provide a source of recommended practices for electrification.

e) Problem areas - Need more people in this subcommittee. Most of our members are track, structural or planning people.

f) Recommend that this group continue their work as it will provide useful information for all electrical properties.

Excerpts From Annual Report of A.R.E.A. Committee 13 - Environmental Engineering*

W. M. Cummings, Chairman

Status of Subcommittee Assignments:

SUBCOMMITTEE 1. WATER POLLUTION CONTROL

C1-2-86. Report on Stormwater Runoff Regulations.

Report was submitted to Railway Track and Structures Magazine and was published in the August, 1992 issue.

D1-3-88. Groundwater Remediation.

All sections, including Bibliography, are complete. Report is in Word Processing and will be submitted for Bulletin publication by December, 1992.

SUBCOMMITTEE 2. AIR POLLUTION CONTROL

D2-1-84. Revision of Manual.

Manual revisions complete and submitted for publication October 5, 1992.

SUBCOMMITTEE 3. LAND POLLUTION CONTROL

D3-1-90. Minimization of Wastewater Treatment Sludges.

Survey has been developed and will be sent to individual railroads in November, 1992. Survey results to be compiled for next Committee meeting in February, 1993.

SUBCOMMITTEE 4. NOISE POLLUTION CONTROL

This Subcommittee remains inactive.

SUBCOMMITTEE 5. PLANT UTILITIES

D5-1-82. Revision of Manual.

Manual revisions were completed and published with the 1992 Manual update.

D5-7-91. Above-Ground Fuel Storage Tank Spill.

Prevention Matrix depicting the various State requirements and agencies involved will be complete and provided to Committee members at the February, 1993 meeting.

Excerpts From Annual Report of A.R.E.A. Committee 14 - Yards and Terminals

R. N. Zimmer, Chairman

Subcommittee A.

This subcommittee functions as a committee of the whole with the context of regular Committee 14 meetings. A number of topics are under consideration for future study. None are being submitted at this time in order that the committee members may concentrate on completing all present assignments. Topics under consideration include:

1. Automatic car identification impacts on yards and terminals.

2. Handling of municipal solid waste.

*At the December 3, 1992 A.R.E.A. Board of Direction meeting, it was voted to disband Committee 13, with assignments and Manual material being distributed to Committees 6, 14 and 27.

3. Yard requirements to support transit, commuter and other passenger operations.

4. Safe control of overspeed cars at hump classification yards with full automatic control.

Subcommittee B.

This subcommittee functions as a committee of the whole with the context of regular Committee 14 meetings. Four study assignments have resulted in proposed revisions to the Manual. Each of these has been reviewed by the committee and approved for letter ballot by the membership of Committee 14. Ballots have been distributed and returned with all four proposals receiving a favorable response.

This subcommittee anticipates three additional study assignments will be completed and additions or revisions to the Manual recommended within the next calendar year. This subcommittee will also be focusing on a detailed review of present Manual material for needed editorial updating.

Subcommittee 1 (D-5-91). Security at Auto Terminals

Subcommittee has visited several auto terminal facilities to collect data on present security measures. A first draft report of their conclusions has been prepared and circulated among subcommittee members. The assignment is rated at a 50% completion level, and should be complete by late 1993. There are no identifiable problems. The committee effort will result in the development of new Manual material.

Subcommittee 2 (C-1-90). Design of Double Stack Facilities

This study assignment has been completed. The resulting proposed addition to the Manual has been submitted by the subcommittee to the full committee for review and approval. The full committee has approved the material submitted for a letter ballot. The results of the letter ballot process was favorable by each measurement method. The proposed addition to the manual has been forwarded to headquarters for additional processing and approvals.

Subcommittee 3 (D-8-88). Bulk Fluid Transfer Facilities

This subcommittee has suffered from the lack of leadership availability this year. The current subcommittee chairman has been severely restricted in the amount of time he can expend on this assignment. The assignment involves the preparation of new Manual material and several drafts have previously been reviewed at the subcommittee and committee levels. With the completion of four other assignments this year, the committee has charged itself with the task of picking up on this assignment and completing it in the upcoming calendar year.

Subcommitee 4 (D3-84). Run Through Trains Effects on Yards

This study assignment has been completed. The resulting proposed addition to the Manual has been submitted by the subcommittee to the full committee for review and approval. The full committee has approved the material submitted for a letter ballot. The results of the letter ballot process was favorable by each measurement method. The proposed addition to the manual has been forwarded to headquarters for additional processing and approvals.

Subcommittee 5 (D-4-86). Control of Cars with Contaminated Wheels

This subcommittee has tried for several years to enumerate the extent of the problem which results in the glazing of retarder shoes and subsequent loss of their effectiveness. Data on such events has not been forthcoming from the operators of automated hump yards. The committee has reviewed the situation and voted to request dropping this assignment. In its place the committee is discussing a possible new assignment to address handling the overspeed cars that result from the ineffectiveness of the glazed retarder shoes.

Subcommittee 6 (C-2-88). Working with 'TRB' on Intermodal Terminals

This is an on-going effort. Three members of Committee 14 currently also hold membership on TRB Committee A2M03 Intermodal Freight Terminal Design. This liaison has proved to be a source for much of the material used in subcommittee 2's assignment. With the continuing emphasis on seamless intermodal goods movement, and TRB's efforts to expand research in the area of rail transportation, this subcommittee will continue to provide meaningful information.

Subcommittee 7 (D-1-90). Fueling and Sanding Facilities in Yards

This study assignment has been completed. The resulting proposed addition to the Manual has been submitted by the subcommittee to the full committee for review and approval. The full committee has approved the material submitted for a letter ballot. The results of the letter ballot process was favorable by each measurement method. The proposed addition to the manual has been forwarded to headquarters for additional processing and approvals.

Subcommittee 8 (D-7-87). Yard Control Systems

This study assignment has been completed. The resulting proposed addition to the Manual has been submitted by the subcommittee to the full committee for review and approval. The full committee has approved the material submitted for a letter ballot. The results of the letter ballot process was favorable by each measurement method. The proposed addition to the manual has been forwarded to headquarters for additional processing and approvals.

Subcommittee 9 (D-2-89). Rail/Water Transfer Facilities

This subcommittee has suffered from the loss of its original chairman. The new subcommittee chairman has made several attempts to advance this assignment. However, many of the members of this subcommittee have been concentrating on the completion of other subcommittee assignments. The proposed output of this subcommittee is a significant expansion of Manual material. One of approximately seven sub-sets of material has been effectively completed. With the completion of four other assignments this year, the committee will be more able to concentrate on this assignment and complete it in the upcoming calendar year.

Excerpts From Annual Report of A.R.E.A. Committee 15 - Steel Structures

M. Noyszewski, Chairman

Brief status of subcommittee assignments:

Subcommittee A. Recommendations for Further Study and Research

Our committee members are continuing to work closely with the AAR/NSF Bridge Research Program through John Choros of the AAR Technical Center. The Bridge Research Steering Committee for this program includes 6 members of Committee 15.

Subcommittee B. Revision of Manual

The Commitee has been working to prepare this year's submittal for changes to Chapter 15 of the AREA Manual and is working closely with other subcommittees on proposed future changes. Parts 1 and 2 will be combined in the 1993 changes.

D1-1-87. Obtain data from which the frequency of occurrence of maximum stress in steel railway bridges may be determined under service loading.

This assignment is being progressed by Subcommittee 1 — Design Loading and Stresses. This subcommittee is working with the rating subcommittee and as the results of the current AAR/NSF Bridge Research Program are broadened to cover a sufficient number and variety of structures. The results will be put into a preliminary document. The analytical data (Scott Beisler's report) is ready but will not be issued until sufficient field corroboration is available.

C2-1-82. Review and update fracture control plan

This assignment is being progressed by Subcommittee 2 — Materials, and the appropriate elements are now in the AREA Manual. However, the fracture control plan requires updating as additional data becomes available.

C3-1-86. For steel fabrication develop materials, methods, quality control procedures and qualifications of fabricators.

This assignment now is under Subcommittee 3 — Fabrication and Erection. The committee continues to work on the development of specification for the loading details for fabricated members, updating fabrication and erection specifications to reflect current practices, and other recommended changes in the area of fabrication and erection.

D6-1-88. Develop methods for repairing damaged steel bridge members.

This assignment is being progressed by Subcommittee 6. Additional presentations have been made during the past year. It is estimated that at least two more years will be necessary to complete this assignment.

D7-1-87. Develop specifications for the design of confined elastomeric bearings in collaboration with Committee 8.

This assignment is being progressed by Subcommittee 7. The subcommittee continues to develop a comprehensive specification for elastomeric bearings with the goal of incorporating this specification into the proposed new Chapter 19 — Bridge Bearings.

It is requested that this assignment be replaced by a new assignment titled "Develop New Chapter 19 — Bridge Bearings" in collaboration with Committees 7 and 8.

C8-1-60. Develop bibliography and technical explanation of various requirements in AREA specifications relating to iron and steel structures.

This assignment is being progressed by Subcommittee 8. There is a continuing need for this committee to update and add items in its area of responsibility.

Excerpts From Annual Report of A.R.E.A. Committee 16 - Economics of Plant, Equipment and Operations

M. W. Franke, Chairman

Report on Special (D) and Continuing (C) Assignments

1. Assigned Number	**B**	**D-2-87**	**D-3-92**	**D-4-92**
2. Description of Assignment	Manual Revision	Artificial Intelligence	Economics of Train Delay	Utiliz. of High Perform. Locos.
3. Date Assigned/ (Undertaken)	Ongoing	1987	1992	1992
4. Estimated % Completion	Ongoing	95%	5%	5%

5. Estimated Completion Date	Ongoing	10/92	12/93	12/93
6. Problems	None	None		
7. Committee Priority Number	*			
8. Recommend Assign. Being Dropped/ Discontinued	No	No	No	No
9. Recommend Assign. Being Postponed	No	No	No	No

***Chapter 16 — Parts 1, 2, 3, and 4 being currently reviewed.**

Excerpts From Annual Report of A.R.E.A. Committee 17 - High Speed Rail

A. E. Shaw, Jr., Chairman

Status of Subcommittee Assignments:

Subcommittee D-1-89. Corridor Evaluation

A. Assigned Jan. 26, 1989

B. The Subcommittee has developed and drafted an outline covering its subject. During the summer, the outline was submitted to entire committee for review.

C. Outline is first stage of subcommittee's assignment and this should be completed by summer of 1993.

D. The benefit will be a standard of evaluating criteria which presently does not exist for North America.

E. No problem areas.

F. Continue activity.

Subcommittee C-1-89. Track, Structures, and Track Train Interactions.

A. Assigned January 26, 1989

B. The Subcommittee has developed and drafted an outline covering its subject. During the summer, the outline was submitted to entire committee for review.

C. Outline is the first stage of subcommittee's assignment and this should be completed by summer of 1993.

D. The benefit will be a standard of evaluating criteria which presently does not exist for North America.

E. No problem areas.

F. Continue activity.

Subcommittee C-2-89. Vehicles, Control, Propulsion System Considerations for High Speed Rail, Collaborating on Electriifcation with Committee 33 — Electrical Energy Utilization, and the AAR Communication and Signal Division

A. Assigned January 26, 1989

B. The Subcommittee has developed and drafted an outline covering its subject. During the Summer, the outline was submitted to entire committee for review.

C. Outline is first stage of subcommittee's assignment and this should be completed by summer of 1993.

D. The benefit will be a standard of evaluating criteria which presently does not exist for North America.

E. No problem areas.

F. Continue activity.

Subcommittee D-2-89. Evaluation of Systems for Operation over 250 M.P.H.

Not assigned for study yet.

Excerpts From Annual Report of A.R.E.A. Committee 18 - Light Density and Short Line Railways

R. A. Paul, Chairman

Status of Subcommittee Assignments:

Subcommittee B

B-1. Review the current manual for its application to short lines.

The committee will complete an organization plan for Chapter 18 and enter the first material in the Chapter in 1993.

B-2. Examine deleted portions of the manual that may be valuable to short lines.

The purpose of this assignment is to assure that historical material is not lost which may be of unique interest to light density or short lines, although it may have been deleted from other chapters of the Manual. A review of historical Manual indexes and articles has been completed and will be reviewed and recommended for closure in early 1993. Little information of current use was found in the review because many historical recommended practices relied on economic assumptions which are no longer valid, and changes in the legal environment have assured that much of the historical material on recommended contracting procedures is not applicable to today's railroading.

A second initiative under this Assignment has been to catalogue technical information on rail sections of 100 pounds and less. Information on these sections is no longer maintained in other Manual chapters. A database has been compiled of dimensions and other information on 186 rail sections. The data is now being reviewed and the first version of the database will be distributed in 1993. The database will be an ongoing committee activity.

C1-1-90. Recommend practices concerning use of second hand track material, including inventory of rail and OTM 100 and less.

A survey of surplus lightweight track materials was completed and distributed to Committee members. A further survey is planned to determine the kinds and quantities of light section rail and other track materials now in place. This information will assist railroads to maintain track which has light rail sections, by indicating potential availability of replacement rail or OTM.

C1-2-90. Recommend practices for obtaining, maintaining, organizing, and operating maintenance of way work equipment on shortlines.

Work on this assignment is just beginning. A bibliography of available materials on the subject will be compiled as a first step in approaching the assignment.

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

The committee is preparing a checklist of factors to be considered when contracting for four typical services: surfacing and ballasting, construction of new track, relaying rail and turnouts, and tie renewals. The committee is also collecting sample contract documents which cover these basic services.

D2-2-90. Investigate inspection criteria for shortlines.

Work on this assignment initially focused on bridge inspection. A draft "Track Inspector's Guide to Bridge Inspection" has been prepared and is under review. This document is nearing completion and will be considered soon for formal inclusion in the AREA Manual. A draft checklist for basic track inspection has also been prepared and is being circulated for comment.

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

A list of representative types of documents has been compiled, and samples from various railroads are being collected. Documents include track charts, grade crossing inventories, track usage agreements, from several committee members. Compilation of these samples into "generic" samples will be completed in 1993. This information is expected to be particularly useful as a guideline for what should be available when sections of railroad change ownership.

D2-4-92. Develop recommended practice for budgeting and planning engineering work of short line and light density lines, collaborating with Committee 22.

A copy of the ICC Statement of Accounts applicable to engineering work has been furnished as a starting point for organizing the approach to this ongoing task.

Excerpts From Annual Report of A.R.E.A. Committee 22 - Economics of Railway Construction & Maintenance*

S. H. Morrell, Chairman

Status of Subcommittee assignments:

Subcommittee A. Revision of Manual

(a) Assigned: Unknown

(b) Progress: Rewrite of entire Manual underway with progress of each part shown below.

(c) Completion date: 6/92 per Committee's five year goals.

*At the December 3, 1992 A.R.E.A. Board of Direction meeting, it was voted to disband Committee 22, with assignments being transferred to Committee 16. A.R.E.A. Manual Chapter 22 will now be maintained by Committee 16.

(d) Benefits: Greatly improved quality of information in Chapter 22 of Manual.

(e) Problems: None. Parts 1, 2 and 4 are complete and were included in the 1991 Manual Update. Part 3 is in progress.

(f) Recommendations: Continue.

Subcommittee B-1, Part 1.
Revisions of Manual - Personnel

(a) Assigned: 1/14/88.

(b) Progress: 100% - Committee approved by ballot.

(c) Completion date: 1/90.

(d) Benefits: Improved information.

(e) Problems: None.

(f) Recommendations: Continue.

Subcommittee B-1, Part 2.
Revisions of Manual - Programming Work and Budgeting

(a) Assigned: 1/14/88.

(b) Progress: 100% - Committee approved by ballot.

(c) Completion date: 1/90.

(d) Benefits: Provide high quality information.

(e) Problems: None.

(f) Recommendations: Continue.

Subcommittee B-1, Part 3.
Revisions of Manual-Construction and Maintenace Operations

(a) Assigned: 1/14/88.

(b) Progress: 95% - Reviewing proposed changes.

(c) Completion date: 1/93.

(d) Benefits: Improved information.

(e) Problems: Information requires further review by experts in the field.

(f) Recommendations: Continue.

Subcommittee B-1, Part 4.
Revisions of Manual - Equated Mileage Parameters

(a) Assigned: 1/14/88.

(b) Progress: 100% - Committee approved by ballot.

(c) Completion date: 1/90.

(d) Benefits: Improved usefulness and understanding of equated mileage parameters.

(e) Problems: None

(f) Recommendations: Continue.

Subcommittee D-1-90. Renewal of Concrete Ties complete or in part.

(a) Assigned: 1/29/90.

(b) Progress: 0% - Reviewing Committee 10's work.

(c) Completion date:

(d) Benefits: Addresses an important and extensive topic.

(e) Problems: None

(f) Recommendations: Continue

Subcommittee D-3-88. Economics of Vegetation Control Methods Working with Committees 1 and 13.

(a) Assigned: 1/14/88.

(b) Progress: 10%.

(c) Completion date:

(d) Benefits: Develop costs of Vegetation Control Methods.

(e) Problems: Change in chairman slowed progress. Reviewing prior efforts to determine whether to start over or continue based on prior information.

(f) Recommendations: Continue.

Subcommittee D-4-90. Track Time Usage

(a) Assigned: 1/29/90

(b) Progress: 30%. Further clarification of the subcommittee's goals is needed.

(c) Completion date:

(d) Benefits: Addresses economics of gang size and track time relationships.

(e) Problems: None.

(f) Recommendations: Continue.

Subcommittee D-6-90. Centralized versus Decentralized Renewal Planning

(a) Assigned: 1/29/90.

(b) Progress: 10%

(c) Completion date:

(d) Benefits: Provide planning alternatives based on size, budget or other economic considerations.

(e) Problems: None.

(f) Recommendations: Continue.

Excerpts From Annual Reports of A.R.E.A. Committee 24 — Engineering Education

R. G. McGinnis, Chairman

Brief status of each subcommittee assignment:

Subcommittee A. Recommendation for further study and research

Assigned: Prior to 1983

Progress: Subcommittee is involved in establishing subcommittee goals for the future years and is responsible for identifying topics that should be handled by the Committee.

Completion Date: Ongoing

Benefits: Provides direction for the committee

Problems: None

Recommendation: Continue

Subcommittee 1. Continuing Education

C1-1-71. Continuing Education

Assigned: Prior to 1983

Progress: Currently organizing a symposium on "Preparation of Railroad Bridges for 125-ton Cars" to be held following the Annual Technical Conference in Chicago, March 1993. The subcommittee is also investigating topics for 1994's continuing education program.

Completion Date: Ongoing

Benefits: Provides opportunities for AREA members to improve their knowledge of subject matters which are relevant to the railroad industry.

Problems: None

Recommendation: Continue

Subcommittee 2. Student Relations

C2-1-82. Recruiting of Engineering Graduates

C2-2-88. Student Interest Award Program

C2-3-71. Student Relations

Assigned: Prior to 1983

Progress: Historically has published an annual survey of college graduate hiring by railroad engineering and maintenance departments. Administers the Student Interest Award Program (SIAP); and oversees the Student Affiliate program.

Completion: Should be continuous

Benefits: Participation in the SIAP increased significantly in 1991.

Problems: The most recent graduate recruiting survey, based on a return of 9 of 18 questionnaires indicated that only 1 graduate was hired by the railroad industry in 1991. Repeat efforts to improve this data have failed due to poor response from the industry. This survey will no longer be conducted by the committee. The Student Affiliate Program continues to have low numbers of members. Increased attention from the Committee is needed.

Recommendations: Continue

Subcommittee 3. Faculty Relations

C3-1-85. Faculty Relations

C3-2-82. Speakers for Student Groups

Assigned 1985

Progress: Speakers at the AREA Technical Conferences are being contacted to see if they can furnish copies of their presentations to be made available to professors.

Completion: Should be continuous

Benefits: Teaching aids for faculty are being assembled

Problems: None

Recommendations: Continue

Excerpts From Annual Report of A.R.E.A. Committee 27 — Maintenance of Way Work Equipment

J. L. Condon, Chairman

Subcommittee assignments:

Subcommittee A: Recommendations for Further Study and Research

The task of this Subcommittee is to provide direction and assistance for the Committee involving new tasks, setting goals and interfacing with the REMSA Group.

Subcommittee B: Revision of Manual.

A Manual revision is currently being submitted for inclusion in the 1993-94 AREA Manual which is an effort to specifically identify colors that machine component or areas must be painted. This will allow manufacturers to reduce paint inventories and result in more uniform and consistent colors being applied to the various Roadway Work Equipment Machines.

Subcommittee 1: Reliability Engineering as Applicable to Work Equipment.

Previous distribution was made of a Committee 27 Work Sheet addressing the overall condition of which new or reconditioned Roadway Work Equipment Machinery has been received by the Railroads from manufacturers. This document is beginning to generate good supportive documentation regarding the condition of equipment as it is received by various Railroads, reflecting the general quality of manufacturing and assembly process which directly relates to the reliability of the machines as they are commissioned for work in the field.

Subcommittee 2: Preventative Maintenance for Maintenance of Way Work Equipment.

Information was distributed to the Committee regarding the use of unleaded gasoline in small engines as typically used in the Maintenance of Way Work Equipment Fleet and the potential benefits and problems that could be the result of using unleaded gasoline in these various type small engines.

Due to the complexity associated with implementing a comprehensive PM system, there has been a variety of discussions and input suggestions as to the direction this group should follow. Some of the previous information and forms that were submitted, were felt to be too complex for the current day average work group and is being suggested that the previously furnished 'PM Directional Preces' be condensed.

This Committee is presently regrouping through previous suggestions and input from the members and will refocus on how to most efficiently and effectively address the preventive maintenance systems relative to Work Equipment.

It was disappointing to learn that the AREA Board questioned the feasibility of the maintenance data requirement (MDR) questionnaire submitted on August 7, 1992, and apparently has been requested to be tabled until the subject can be discussed with vendors who may later join Committee 27. Unfortunately, these are precisely the types of information required from the manufacturers to effectively pursue the PM issues. Further, this is one of the contributing reasons Subcommittee 2 will have problems progressing their agenda in a timely fashion.

Subcommittee 3: Computer Applications within the Work Equipment Organization.

This Committee has regrouped with a new Chairman being established. Initial direction, goals and agenda of topics have been established. This group has a renewed enthusiasm and we should see some very worthwhile results in the very near future. Some of the topics consist of:

1. Using electronic mail bulletin boards on an industry-wide basis for service bulletins and equipment modification information.

2. Equipment on-board computers and micro-processors for monitoring maintenance and production information should be defined in a format with considerations given to hardware, software, networking, etc.

3. General Data Communication Networking.

4. Electronic Data Transfer relating to Purchasing and Material Acquisition Systems.

Subcommittee 4: Maintenance of Way Work Equipment Safety and Ergonomics.

Various issues have been actively discussed within the Committee including:

1. Noise levels for the various types of machines.
2. Individual roads hearing protection practices relating to the equipment noise generation.
3. Prevention of noise through feasible design changes.
4. Sound Nullification Systems.
5. Operator Ergonomics and new seats available for operator stations.
6. Crane safety problems and methods of inspection to resolve these problems.
7. Lockout/tagout procedures.
8. Machine guarding practices.

Subcommittee 5: Training Programs for Machine 0perators and Maintenance Personnel.

All Railroads are currently being requested to furnish any available lists or inventories of operator/maintenance training videos which could be acquired by any Railroad. The effort to exchange training programs has all but lost interest because various Railroads are fearful of furnishing for the reason of:

1. Liability associated with furnishing specific training information to other Railroads.
2. Lack of reciprocal exchange of training programs from other Railroads.
3. A request has been sent to the manufacturers for any available operator and maintenance and safety videos by equipment type and machine model numbers and the respective acquisition cost. Further, it has been discussed that a standard format for training videos should be incorporated into the Committee 27 Manual as well as establishing a training video library or catalogue relative to Roadway Work Equipment Machinery.

Excerpts From Annual Report of A.R.E.A. Committee 28 - Clearances

K. W. Eich, Chairman

Subcommittee Progess:

Subcommittee A. Recommendations for Further Study and Research

An on-going subcommittee to develop future studies has recently recommended the newest subcommittee on Double Stack Containers. It is this subcommittees responsibility to investigate thoroughly areas of future concern to us as well as the organization and industry.

Subcommittee B. Revision of the Manual.

Currently in the process of revising the Legal Clearance Requirements sheet of the various state entities within the chapter. More revisions will follow upon completion of this current assignment.

C-1-62. Compilation of the railroad clearance requirements of the various states.

As mentioned in the above, both subcommittees are working together toward completion of this project, which is intended for committee presentation at the spring meeting.

C-2-85. Compilation of a comprehensive glossary and bibliography pertaining to the technical literature on railroad high and wide clearances.

This glossary is a point of reference for railroads and shippers alike. They are coming together to have a common understanding of the technical terms that we have been defining and use on a daily basis. These definitions are also found in the Railway Line Clearances publication.

C-3-85. Review of *Railway Line Clearances* to develop improved user accessibility.

In the past year, a number of issues have been addressed. If successfully resolved, future utility of the publication can be improved. Several of these issues are; reducing the number of clearance columns, enlarging the print size, clarification or elimination of the map representations, expand the supplied information to include heavy duty car capabilities, etc.

C-4-91. Research, report and provide equipment clearance diagrams (plates) as required."

Currently in the process of developing a new diagram for double stack containers and tri-levels for submittal to the AAR. In addition, this subcommittee had presented three (3) scenarios to the AAR "Future Distributions Systems Task Force" for review of equipment needs of the future.

C-5-92. Monitor and report on clearance changes within the Double Stack Container (COFC) industry."

This subcommittee has just been "commissioned" but indicates that it will provide a great deal of assistance in the areas of the ever increasing dimensions of equipment such as Double Stacks, Tri-Levels and the such.

D-3-85. Conversion of 'Heavy Capacity and Special Type Flat Car' section of *The Official Railway Equipment Register* to UMLER compatible format.

It appears that this subcommittee has done it's job and we are taking one close last look prior to putting the subcommittee to sleep. We expect the final report at the spring meeting.

D-4-85. Research and develop book covering heavy-duty car diagrams and ratings.

The last publication of any book such as this was by the AAR back in the late 60's or early 70's. The book will be beneficial to any and all individuals who deal with excessive weights on heavy-duty cars that traverse the rail corridors of North America. One railroad has yet to submit the last car of information in order to complete the subcommittee.

D-6-89. Recommendations for a uniform electronic clearance message.

This subcommittee is complete. The report was published in A.R.E.A. Bulletin No. 737 (October '92). At that time, we requested headquarters submit a copy of the report to the AAR's EDI committee for a response. As soon as some type of reply is received, this subcommittee will have successfully completed it's assignment.

Excerpts From Annual Report of A.R.E.A. Committee 32 - Systems Engineering

D. E. Bartholomew, Chairman

Subcommittee Progress:

Subcommittee A. Recommendations for further study and research.

Subcommittee has investigated changes and additions to all subcommittee assignments and has reviewed the progress of the subcommittee developing Manual material. No change in subcommittee assignments is now recommended.

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

C2-1-86. Research new applications for the use of information technology that will assist in the management of engineering resources.

This is an ongoing assignment. The subcommittee has prepared a survey of existing and proposed applications of the various member roads and is currently reviewing the results. Subcommittee research results is usually disseminated via Committee sponsored Symposiums, presentations at Committee Meetings, Proceedings Articles, and March Conference Presentations. Research results also are forming a substantial base for the development of Manual Material.

D2-1-92. Initiate the development of Manual Material for Engineering Management Systems.

Progress has been made on the development of a Table of Contents for Manual Chapter 32, which will serve as an outline for future tasks. This will be followed by development of a general introductory, recommendation and conclusion section which will include a general high level Flow Diagram of recommended Engineering Systems and Applications.

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

The subcommittee sponsored a presentation at the March 1992 Conference.

D3-1-92. Present a symposium in 1993 on Engineering Systems.

Work on preparing a Symposium in conjunction with the B&B and Road Masters Conference and REMSA Show in Denver is progressing.

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

D4-1-86. Develop a Manual section with recommended protocol for CADD drawings.

Work is continuing on this assignment. The subcommittee has disseminated surveys to determine the types of CADD equipment and systems used on the various roads. Several CAD drawings, including some of the initial drawings prepared by the consultant for the "Portfolio of Trackwork Plans," have been transmitted between several railroads and tested to determine the effectiveness of transfer techniques and protocols.

Due to the rapid advance in technology this investigation and the survey of the various railroads must be updated every few years. It is very difficult to make recommendations without favoring a specific popular equipment vendor. The subcommittee is, however, making progress toward developing a generic, albeit very general, recommendation. They are also looking at putting together a catalog of CADD specific information and reports and are investigating how various state highway departments are handling the standards problem.

The subcommittee has conducted a number of tests of the CAD drawings developed for inclusion in the "Portfolio of Trackwork Plans." There have been a number of informal discussions with members of Committee 5. We now have a joint member of Committee 5 and 32 that is also employed by the consultant preparing the CAD drawings.

Excerpts From Annual Report of Committee 33 - Electrical Energy Utilization

J. Popoff, Chairman

Status of Subcommittees:

Subcommittee 1. Electrification Economics

Recent pricing information from the award of the New Haven to Boston electrification is providing current and accurate data for an update of Part 1 of Chapter 33 of the AREA Manual.

Subcommittee 4. Railroad Electrification Systems

The bulk of the Committee's activities have been concentrated in this subcommittee. The major advances in railroad electrification worldwide have taken place in this area. Part 4 of Chapter 33 of the AREA Manual has been completely reviewed and is in the final process of a complete update. One of the major components of the update is the matter of electrical clearances. With the resurgence of interest in electrification in North America it has been necessary to update the clearance envelope necessary to accommodate electrification on existing rail lines. Standards used in Europe and elsewhere in the world have been reviewed and incorporated. The final draft of the proposed language should be ready for presentation and discussion at the general committee meeting in March.

Subcommittee 6. Power Supply and Distribution

The updating of Part 6 is in progress and is approximately 60% complete. Work has been substantially slowed as a result of concentration on completion of Part 4. It is expected that Part 6 activities will increase after March 1993.

Subcommittee 8. Equipment Generated Electrical Noise

The Committee has elected to hold work on Part 8 in abeyance until the upates of Parts 4, 6, and 1 (in turn) are completed.

Excerpts From Annual Report of A.R.E.A. Committee 34 - Scales

W. G. GeMeiner, Chairman

Activities of Subcommittees:

Subcommittee A. Recommendations for Further Study and Research

This is an ongoing subcommittee that investigates and reviews new weighing devices and technological advances in the Weighing Industry; making recommendations to the Committee Chairman concerning need for further study and research. The subcommittee is recommending that a metrification study be started, possibly in 1993 after other subcommittee work is finalized.

Subcommittee B. Revisions of the AAR Scale Handbook

This is an ongoing subcommittee that revises the AAR Scale Handbook as necessary, on an annual basis, so that it remains up-to-date with technological advances and compatible with State and Federal regulations. The subcommittee chairman is coordinating a detailed review of the entire Handbook, since it is felt that scale installation guidelines should be updated to reflect the fact that most new scales are electronic rather than mechanical. This subcommittee is one of the most important within Committee 34. Installation and performance requirements need continual review to keep current with technology. This subcommittee will be resized (smaller) to improve effectiveness.

C-1-85. Preparation of Subjects for Publication

This is an ongoing subcommittee that does final editing of items to be published as information in the AREA Bulletin or as recommended practice in the AAR Scale Handbook. There are currently no outstanding assignments. A recommendation from Subcommittee D-2-87 may be published in the AREA Bulletin in the Fall of 1993, subsequent to a final vote on C-I-M weighing regulations pending at the National Conference on Weights and Measures.

C-2-82. Innovations in Scales Used in Connection with Operations of Railroads

This is an ongoing subcommittee that investigates new technology and changes in the weighing industry that affect railroads. The subcommittee determines the relevance and suitability of new technologies, pertaining to their application and cost effectiveness in a railroad environment. They have recently reported on the new compression-style digital load cells used in both static and and motion weighing track scales. The subcommittee will soon be evaluating a high speed motion weighing system on the BN Railroad in Kansas City that utilizes strain gauges bonded to the running rail.

D-1-83. Criteria for the Location of Coupled-in-Motion Track Scales

The final draft of this study was published in the December 1990, AREA Bulletin. A number of subcommittee members are associated with the Railroad Advisory Committee, who has been involved with the NCWM in a study of coupled-in-motion weighing systems that are used in individual car mixed-manifest custody transfer and tank car weighing applications. The subcommittee will continue to work with the S&T Committee of the NCWM to evaluate data from the study. The NCWM will be voting on these issues in July - 1993 in Kansas City.

D-2-87. Investigate Stencilling of Cars Using Coupled-in-Motion Weights

This subcommittee's assignment is to determine whether coupled-in-motion weighing systems are suitable when weighing empty railroad cars for the purpose of updating UMLER and/or for the stencilling of cars. If suitable, they will recommend guidelines for implementation. The subcommittee has developed a recommendation that has been submitted to the full Committee for vote and carried. The subcommittee, however, has requested that the recommendation be withheld from publishing in the

AREA bulletin due to some of the voting items (concerning C-I-M) that are being deliberated by the NCWM in1993. We anticipate that the recommendation may be ready for publication as soon as July 1994, depending on the circumstances surrounding the previously mentioned items.

D-3-90. Track Scale Testing Guidelines, Test and Inspection Forms.

This subcommittee is evaluating track scale test and inspection forms in use on various railroads; with the intent of recommending a uniform report format. A draft form, with usage recommendations has been prepared. The subcommittee has had difficulty in reaching consensus in several key areas. Because of the perceived importance of the uniform test and inspection form format, the subcommittee will redouble it's efforts to publish an informational bulletin report by the Fall of 1993.

D-4-91. Railroad Master Scale Program

This subcommittee is studying the existing railroad master track scale program in a number of areas; including age, location, capacity, utilization and general condition, to determine if the "Program" continues to meet the Industries' needs. Engineering specifications have been received from most of the master scale owners; as well as calibration data from the Federal Grain Inspection Service (FGIS), the governmental agency that tests all raiload master scales. They will study the effects of the newer ram-style and heavier scale test cars on the existing scales, and determine if special procedures are needed to protect these older scales. The subcommittee is investigating the alternatives to existing master scale design, including the use of electronics.

Proposed 1993 A.R.E.A. Manual and Portfolio Revisions

The following proposed Revisions of the A.R.E.A. *Manual for Railway Engineering and Portfolio of Trackwork Plans* have been recommended to the association by the technical committee responsible for each after a letter ballot is approved by: (1) a two-thirds majority of the eligible members voting, and (2) by at least fifty percent of the total eligible voting members on the committee. They are being published here for comment by the general A.R.E.A. membership and any other interested parties. Comments should be sent to A.R.E.A. headquarters by March 1, 1993. These comments will be considered by the A.R.E.A. Board of Direction in deciding whether to give final approval for inclusion of the proposed changes in the Manual and Portfolio Revisions, which if approved, go into effect August 1, 1993.

Proposed 1993 Manual Revisions To Chapter 1 - Roadway and Ballast

Part 5 — Pipelines of Chapter 1 has been rewritten to include a new specification for uncased gas pipelines within the railroad right of way. Also included in the rewrite was a review and updating of the existing Part 5 on Specifications for Pipelines Conveying Flammable Substances (retitled to Specifications for Pipelines Conveying Liquid Flammable Substances), and Specifications for Pipelines Conveying Non-Flammable Substances. The new Part 5 proposed is as follows:

Part 5

[1]Pipelines

1993

5.1 SPECIFICATIONS FOR PIPELINES CONVEYING LIQUID FLAMMABLE SUBSTANCES

5.1.1 Scope

These specifications cover minimum requirements for pipelines installed on or adjacent to railway rights-of-way to carry liquid flammable products or highly volatile substances under pressure. The term "engineer" used herein means the chief engineer of the railway company or his authorized representative. These specifications may be increased when risks from any of the following conditions are increased;

1) Track speed
2) Traffic density
3) Traffic sensitivity
4) Terrain conditions, cuts/fills, etc.
5) Curvature and grade
6) Bridges and other structures
7) Pipe size, capacity and material carried
8) Environmental risks/damages

5.1.2 General Requirements

Pipelines under railway tracks and across railway rights-of-way shall be encased in a larger pipe or conduit called the casing pipe as indicated in Fig. 5.1.2.1. Casing pipe may be omitted in the following locations:

a) Under secondary or industry tracks as approved by the engineer.
b) On pipelines in streets where the stress in the pipe from internal pressure and external loads does not exceed 40 percent of the specified minimum yield strength (multiplied by longitudinal joint factor) of the steel pipe material.
c) On gas pipelines as provided in Article 5.2.

Pipelines shall be installed under tracks by boring or jacking, if practicable.

Pipelines shall be located, where practicable, to cross tracks at approximately right angles thereto but preferably at not less than 45 degrees and shall not be placed within a culvert, under railway bridges nor closer than 45 ft. to any portion of any railway bridge, building or other important structure, except in special cases and then by special design as approved by the engineer.

Pipelines carrying liquefied petroleum gas or natural gas liquids shall, where practicable, cross any railway where tracks are carried on an embankment.

Emergency response procedures should be developed to handle a situation in which a pipeline leak or railroad derailment or incident may jeopardize the integrity of the pipeline. Local conditions should be considered when developing these procedures.

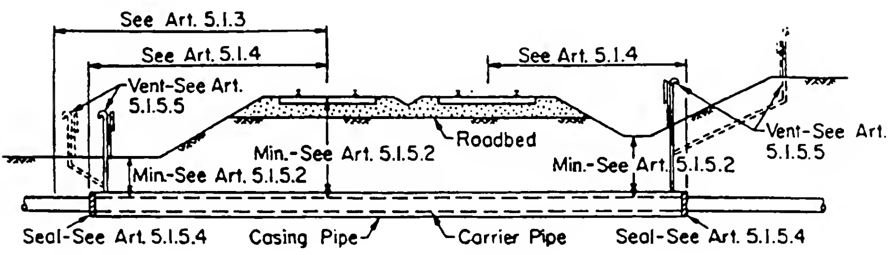


Figure 5.1.2.1

Any replacement of a carrier pipe shall be considered a new installation, subject to the requirements of these specifications.

Where laws or orders of public authority prescribe a higher degree of protection than specified herein, then the higher degree of protection so prescribed shall be deemed a part of these specifications.

Pipelines and casing pipe shall be suitably insulated from underground conduits carrying electric wires on railway rights-of-way. All pipelines, except those in streets, shall be prominently marked at the rights-of-way (on both sides of track for undercrossings) by signs substantially worded thus:

"High pressure....main.....in vicinity. Call....

Additional signing may be required by the engineer where above signs are not readily visible from the track.

5.1.3 Carrier Pipe

Pipelines carrying oil, liquefied petroleum gas and other flammable liquid products shall be of steel and conform to the requirements of the current ANSI B 31.4 Liquid Transportation Systems for Hydrocarbons, Liquid Petroleum Gas, Anhydrous Ammonia, and Alcohols, and other applicable ANSI codes, except that the maximum allowable stresses* for design of steel pipe shall not exceed the following percentages of the specified minimum yield strength (multiplied by longitudinal joint factor) of the pipe as defined in the above codes.

Requisites for carrier line pipe under railway tracks shall apply for a minimum distance of 50 ft. (measured at right angles) from centerline of outside tracks or 2 ft. beyond toe of slope or 25 ft. beyond the ends of casing (when casing is required), whichever is greater.

The pipe shall be laid with sufficient slack so that it is not in tension.

*If the maximum allowable stress in the carrier pipe on either side of the crossing is less than specified above, the carrier pipe at the crossing shall be designed at the same stress as the adjacent carrier pipe.

5.1.3.1 Allowable Hoop Stress Due to Internal Pressure

5.1.3.1.1 With Casing Pipe

The following percentages apply to hoop stress in steel pipe within a casing under railway tracks and across railway rights-of-way:

(a) Seventy-two percent on oil pipelines.

(b) Fifty percent for pipelines carrying condensate, natural gasoline, natural gas liquids, liquefied petroleum gas, and other liquid petroleum products.

(c) Sixty percent for gas pipelines.

5.1.3.1.2 Without Casing Pipe

The following percentages apply to hoop stress in steel pipe without a casing under secondary or industry tracks:

(a) Sixty percent for oil pipelines.

(b) Forty percent for pipelines carrying condensate, natural gasoline, natural gas liquids, liquefied petroleum gas, and other liquid petroleum products.

(c) For gas pipelines see Section 5.2.

5.1.3.1.3 On Right-of-Way

The following percentages apply to hoop stress in steel pipe laid longitudinally on railway rights-of-way:

(a) Sixty percent for oil pipelines.

(b) Forty percent for pipelines carrying condensate, natural gasoline, natural gas liquids, liquefied petroleum gas, and other liquid petroleum products.

(c) For gas pipelines see Section 5.2

5.1.4 Casing Pipe

Casing pipe and joints shall be of steel and of leakproof construction, capable of withstanding railway loading. The inside diameter of the casing pipe shall be such as to allow the carrier pipe to be removed subsequently without disturbing the casing pipe. All joints or couplings, supports, insulators or centering devices for the carrier pipe within a casing under railroad tracks shall be taken into account.

When casing is installed without benefit of a protective coating or said casing is not cathodically protected, the wall thickness shall be increased to the nearest standard size which is a minimum of 0.063 in. greater than the thickness required except for diameters under 12-3/4 in.

Table 5.1.4.1
Minimum Wall Thickness for Steel Casing Pipe for E80 Loading

Nominal Diameter (inches)	When coated or cathodically protected Nominal Thickness (inches)	When not coated or cathodically protected Nominal Thickness (inches)
12-3/4 and under	0.188	0.188
14	0.188	0.250
16	0.219	0.281
18	0.250	0.312
20 and 22	0.281	0.344
24	0.312	0.375
26	0.344	0.406

Table 5.1.4.1 (Continued)
Minimum Wall Thickness for Steel Casing Pipe for E80 Loading

Nominal Diameter (inches)	When coated or cathodically protected Nominal Thickness (inches)	When not coated or cathodically protected Nominal Thickness (inches)
28	0.375	0.438
30	0.406	0.469
32	0.438	0.500
34 and 36	0.469	0.531
38	0.500	0.562
40	0.531	0.594
42	0.562	0.625
44 and 46	0.594	0.656
48	0.625	0.688
50	0.656	0.719
52	0.688	0.750
54	0.719	0.781
56 and 58	0.750	0.812
60	0.781	0.844
62	0.812	0.875
64	0.844	0.906
66 and 68	0.875	0.938
70	0.906	0.969
72	0.938	1.000

5.1.4.1 Steel Pipe

Steel pipe shall have a specified minimum yield strength, SMYS, of at least 35,000 psi.

5.1.4.2 Flexible Pipe

For flexible casing pipe, a maximum vertical deflection of a casing pipe of 3 percent of its diameter plus 1/2 in. clearance shall be provided so that no loads from the roadbed, track, traffic or casing pipe itself are transmitted to the carrier pipe. When insulators are used on the carrier pipe, the inside diameter of flexible casing pipe shall be at least 2 in. greater than the outside diameter of the carrier pipe for pipe less than 8 in. in diameter; at least 3-1/4 in. greater for pipe 8 in. to 16 in., incl., in diameter and at least 4-1/2 in. greater for pipe 18 in. in diameter and over.

5.1.4.3 Length of Pipe

Casing pipe under railway tracks and across railway rights-of-way shall extend to the greater of the following distances, measured at right angles to centerline of track. If additional tracks are constructed in the future or the railway determines that the roadbed should be widened, the casing shall be extended or other special design incorporated.

(a) 2 ft. beyond toe of slope.
(b) 3 ft. beyond ditch.
(c) A minimum distance of 25 ft. each side from centerline of outside track when casing is sealed at both ends.
(d) A minimum distance of 45 ft. each side from centerline of outside track when casing is open at both ends.

5.1.5 CONSTRUCTION

Casing pipe shall be so constructed as to prevent leakage of any substance from the casing throughout its length, except at ends of casing where ends are left open, or through vent pipes when ends of casing are sealed. Casing shall be so installed as to prevent the formation of a waterway under the railway, and with an even bearing throughout its length, and shall slope to one end (except for longitudinal occupancy).

Where casing and/or carrier pipe is cathodically protected, the engineer shall be notified and a suitable test made to ensure that other railway structures and facilities are adequately protected from the cathodic current in accordance with the recommendation of current Reports of Correlating Committee on Cathodic Protection, published by the National Association of Corrosion Engineers.

5.1.5.1 Method of Installation

(a) Installations by open-trench methods shall comply with Installation of Pipe Culverts, Part 4, this Chapter.
(b) Bored or jacked installations shall have a bored hole diameter essentially the same as the outside diameter of the pipe plus the thickness of the protective coating. If voids should develop or if the bored hole diameter is greater than the outside diameter of the pipe (including coating) by more than approximately 1 in., remedial measures as approved by the engineer shall be taken. Boring operations shall not be stopped if such stoppage would be detrimental to the railway.
(c) Tunneling operations shall be conducted as approved by the engineer. If voids are caused by the tunneling operations, they shall be filled by pressure grouting or by other approved methods which will provide proper support.

5.1.5.2 Depth of Installation

5.1.5.2.1 Casing Pipe

Casing pipe under railway tracks and across railway rights-of-way shall be not less than 5-1/2 ft. from base of railway rail to top of casing at its closest point, except that under secondary or industry tracks this distance may be 4-1/2 ft. On other portions of rights-of-way where casing is not directly beneath any track, the depth from ground surface or from bottom of ditches to top of casing shall not be less than 3 ft.

5.1.5.2.2 Carrier Pipe

Carrier pipe installed under secondary or industry tracks without benefit of casing shall be not less than 10 ft. from base of railway rail to top of pipe at its closest point nor less than 6 ft. from ground surface or from bottom of ditches.

5.1.5.3 Inspection and Testing

ANSI Codes current at time of constructing the pipeline, shall govern the inspection and testing of the facility within the railway rights-of-way except as follows:

(a) One-hundred percent of all field welds shall be inspected by radiographic examination, and such field welds shall be inspected for 100 percent of the circumference.
(b) The proof testing of the strength of carrier pipe shall be in accordance with ANSI requirements.

5.1.5.4 Seals

Where ends of casing are below ground they shall be suitably sealed.

Where ends of casing are at or above ground surface and above high-water level they may be left open, provided drainage is afforded in such manner that leakage will be conducted away from railway tracks or structure. Where proper drainage is not provided, the ends of casing shall be sealed.

5.1.5.5 Vents

Casing pipe, when sealed, shall be properly vented. Vent pipes shall be of sufficient diameter, but in no case less than 2 in. in diameter, shall be attached near end of casing and project through ground surface at right-of-way lines or not less than 45 ft. (measured at right angles) from centerline of nearest track. Vent pipe, or pipes, shall extend not less than 4 ft. above ground surface. Top of vent pipe shall be fitted with down-turned elbow properly screened, or a relief valve. Vents in locations subject to high water shall be extended above the maximum elevation of high water and shall be supported and protected in a manner that meets the approval of the engineer. Vent pipes shall be no closer than 4 ft. (vertically) from aerial electric wires.

5.1.5.6 Shut-Off Valves

Accessible emergency shut-off valves shall be installed within effective distances each side of the railway as mutually agreed to by the engineer and the pipeline company. These valves should be marked with signs for identification. Where pipelines are provided with automatic control stations at locations and within distances approved by the engineer, no additional valves shall be required.

5.1.5.7 Longitudinal Pipelines

Longitudinal pipelines should be located as far as possible from any track. They must not be within 25 ft. of any track and must have a minimum of 6 ft. ground cover over the pipeline up to 50 ft. from centerline of track. Where pipeline is laid more than 50 ft. from centerline of track, minimum cover shall be at least 5 ft. Pipelines must be marked by a sign approved by the engineer every 500 ft. and at every road crossing, streambed, other utility crossing, and at locations of major change in direction of the line.

5.1.6 Approval of Plans

Plans for proposed installation shall be submitted to and meet the approval of the engineer before construction is begun.

Plans shall be drawn to scale showing the relation of the proposed pipeline to railway tracks, angle of crossing, location of valves, railway survey station, right-of-way lines and general layout of tracks and railway facilities. Plans should also show a cross section (or sections) from field survey, showing pipe in relation to actual profile of ground and tracks. If open-cutting or tunneling is necessary, details of sheeting and method of supporting tracks or driving tunnel shall be shown.

In addition to the above, plans should contain the following data:

	Carrier Pipe	Casing Pipe
Contents to be handled	....	....
Outside Diameter	....	....
Pipe Material	....	....
Specification and grade	....	....
Wall thickness	....	....
Actual Working pressure	....	....
Type of joint	....	....
Coating	....	....
Method of installation	....	....
Vents: Number	Size	Hgt. above ground
Seals: Both ends	one end	Type ....
Bury: Base of rail to top of casing	.... ft.	.... in.
Bury: (Not beneath tracks)	.... ft.	.... in.
Bury: (Roadway ditches)	.... ft.	.... in.
Type, size and spacing of insulators or supports	....	
Distance C.L. track to face of jacking/receiving pits	.... ft.	.... in.
Bury: Base of rail to bottom jacking/receiving pits	.... ft.	.... in.
Cathodic protection	yes	no ....

5.1.7 Execution of Work

The execution of work on railway rights-of-way, including the supporting of tracks, shall be subject to the inspection and direction of the engineer.

5.2 SPECIFICATIONS FOR UNCASED GAS PIPELINES WITHIN THE RAILWAY RIGHT OF WAY

5.2.1 Scope

These specifications cover minimum specifications for pipelines installed on or adjacent to railway rights-of-way to carry flammable and nonflammable gas products which, from their nature or pressure, might cause damage if escaping on or in the vicinity of railway property. The term "engineer" as used herein means the chief engineer of the railway company or his authorized representative. These specifications may be increased when risks from any of the following conditions are increased;

1) Track Speed
2) Traffic density
3) Traffic sensitivity
4) Terrain conditions, cuts/fills, etc.
5) Curvature and grade
6) Bridges and other structures
7) Pipe size, capacity and material carried
8) Environmental risks/damages

5.2.2 General Requirements

Pipelines shall be installed under tracks by boring or jacking, if practicable.

Pipelines shall be located, where practicable, to cross tracks at approximately 90 degrees but not less than 45 degrees, and shall not be placed within a culvert, under railway bridges, nor closer than 45 ft. to any portion of any railway bridge, building or other important structure, except in special cases and then by special design as approved by the engineer.

Pipelines carrying flammable gas products shall, where practicable, cross any railway where tracks are carried on an embankment.

Emergency response procedures should be developed to handle a situation in which a pipeline leak or railroad derailment or incident may jeopardize the integrity of the pipeline. Local conditions should be considered when developing these procedures.

Uncased gas pipelines under railroad track and on right of way shall be installed as indicated in Fig. 5.2.2.1.

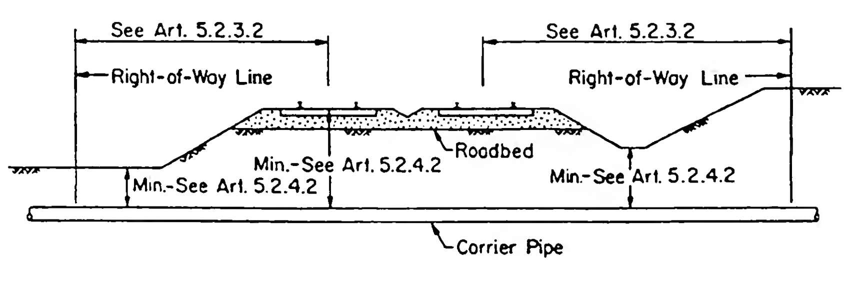


Figure 5.2.2.1

Where laws or orders of public authority prescribe a higher degree of protection than specified herein, then the higher degree of protection so prescribed shall be deemed a part of these specifications.

Pipelines and casing pipe shall suitably insulated from underground conduits carrying electric wires on railway rights-of-way. All pipelines, except those in streets, shall be prominently marked at the rights-of-way (on both sides of track for undercrossings) by signs substantially worded thus:

"High pressure..... main...... in vicinity. Call....

Additional signing may be required by the engineer where above signs are not readily visible from the track.

5.2.3 Carrier Pipe

Pipelines carrying flammable and nonflammable gas products shall be of steel and shall conform to the requirements of the current ANSI B 31.8 Gas Transmission and Distribution Piping Systems, and other applicable ANSI codes.

Carrier line pipe construction shall be approved by the engineer. Joints for carrier line pipe must be of an approved welded type. Steel pipe must have a specified minimum yield strength, SMYS, of at least 35,000 psi. The nominal wall thickness for the steel carrier pipe, specified minimum yield strength, SMYS, maximum allowable operating pressure, MAOP, and outside pipe diameter, D, are given in Tables 5.2.3 (a through j).

These Table wall thicknesses are based on four design criteria. These design criteria consider:

a) The maximum allowable hoop stress due to internal pressure as specified in regulatory codes;
b) The maximum combined multiaxial stress due to all external and internal loads;
c) Fatigue in girth welds due to external live loads;
d) Fatigue in longitudinal seam welds due to external live loads.

The greatest wall thickness resulting from each of the design conditions are shown in the Tables.

Design parameter assumptions used to calculate the Table wall thicknesses are as follows:

Depth of carrier from base of rail is 10 ft.
Double Track condition is assumed
Modulus of Soil Reaction E′ = 500 psi
Soil Resilient Modulus E_r = 10,000 psi
Girth weld is located at centerline of track
Overbore of 2″ over pipe diameter during installation
Class location design factor F = 0.6 used in design criterion a) given above
Factor of Safety FS = 1.5 used in design criteria b), c), and d) given above
See Table 8 in the Commentary for additional details on the design parameters used to determine the wall thicknesses.

If actual crossing conditions fall outside these parameters, tending to require a thicker walled pipe, a detailed analysis must be performed using the design methodology referenced in Part 5.2.8. Design calculations must be provided for railroad review when conditions outside those above are present.

Minimum Nominal Wall Thicknesses (in.) for Uncased Carrier Pipe

	MAOP ≤ 100 psi					MAOP ≤ 200 psi				
	SMYS (psi) ≥					SMYS (psi) ≥				
D (in.)	**35000**	**42000**	**52000**	**60000**	**70000**	**35000**	**42000**	**52000**	**60000**	**70000**
≤ 18.0	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
20.0	0.219	0.219	0.219	0.219	0.219	0.219	0.219	0.219	0.219	0.219
22.0	0.226	0.226	0.226	0.226	0.226	0.226	0.226	0.226	0.226	0.226
24.0	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250	0.250
26.0	0.281	0.281	0.281	0.281	0.281	0.281	0.281	0.281	0.281	0.281
28.0	0.281	0.281	0.281	0.281	0.281	0.312	0.281	0.281	0.281	0.281
30.0	0.312	0.312	0.312	0.312	0.312	0.344	0.312	0.312	0.312	0.312
32.0	0.344	0.344	0.344	0.344	0.344	0.344	0.344	0.344	0.344	0.344
34.0	0.344	0.344	0.344	0.344	0.344	0.406	0.344	0.344	0.344	0.344
36.0	0.375	0.375	0.375	0.375	0.375	0.406	0.375	0.375	0.375	0.375
38.0	0.406	0.406	0.406	0.406	0.406	0.438	0.406	0.406	0.406	0.406
40.0	0.406	0.406	0.406	0.406	0.406	0.469	0.406	0.406	0.406	0.406
42.0	0.438	0.438	0.438	0.438	0.438	0.500	0.438	0.438	0.438	0.438

Table 5.2.3a

	MAOP ≤ 300 psi					MAOP ≤ 400 psi				
	SMYS (psi) ≥					SMYS (psi) ≥				
D (in.)	**35000**	**42000**	**52000**	**60000**	**70000**	**35000**	**42000**	**52000**	**60000**	**70000**
≤ 12.75	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
14.0	0.188	0.188	0.188	0.188	0.188	0.203	0.188	0.188	0.188	0.188
16.0	0.188	0.188	0.188	0.188	0.188	0.281	0.188	0.188	0.188	0.188
18.0	0.219	0.188	0.188	0.188	0.188	0.281	0.219	0.188	0.188	0.188
20.0	0.250	0 219	0.219	0.219	0.219	0.312	0.250	0.219	0.219	0.219
22.0	0.281	0.226	0.226	0.226	0.226	0.344	0.281	0.226	0.226	0.226
24.0	0.312	0.250	0.250	0.250	0.250	0.375	0.281	0.250	0.250	0.250
26.0	0.344	0.281	0.281	0.281	0.281	0.406	0.312	0.281	0.281	0.281
28.0	0.375	0.312	0.281	0.281	0.281	0.438	0.344	0.281	0.281	0.281
30.0	0.406	0.312	0.312	0.312	0.312	0.469	0.375	0.312	0.312	0.312
32.0	0.438	0.344	0.344	0.344	0.344	0.500	0.406	0.344	0.344	0.344
34.0	0.469	0.375	0.344	0.344	0.344	0.531	0.438	0.344	0.344	0.344
36.0	0.500	0.406	0.375	0.375	0.375	0.562	0.469	0.375	0.375	0.375
38.0	0.531	0.438	0.406	0.406	0.406	0.625	0.500	0.406	0.406	0.406
40.0	0.562	0.469	0.406	0.406	0.406	0.656	0.531	0.406	0.406	0.406
42.0	0.594	0.500	0.438	0.438	0.438	0.688	0.562	0.438	0.438	0.438

Table 5.2.3b

Minimum Nominal Wall Thicknesses (in.) for Uncased Carrier Pipe (continued))

	MAOP ≤ 500 psi					MAOP ≤ 600 psi				
	SMYS (psi) ≥					SMYS (psi) ≥				
D (in.)	35000	42000	52000	60000	70000	35000	42000	52000	60000	70000
≤ 8.625	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
10.75	0.188	0.188	0.188	0.188	0.188	0.203	0.188	0.188	0.188	0.188
12.75	0.219	0.188	0.188	0.188	0.188	0.250	0.203	0.188	0.188	0.188
14.0	0.250	0.188	0.188	0.188	0.188	0.281	0.210	0.188	0.188	0.188
16.0	0.281	0.219	0.188	0.188	0.188	0.312	0.250	0.188	0.188	0.188
18.0	0.312	0.250	0.188	0.188	0.188	0.344	0.281	0.219	0.188	0.188
20.0	0.344	0.281	0.219	0.219	0.219	0.375	0.312	0.250	0.219	0.219
22.0	0.375	0.312	0.250	0.226	0.226	0.438	0.344	0.281	0.226	0.226
24.0	0.406	0.344	0.281	0.250	0.250	0.469	0.375	0.312	0.250	0.250
26.0	0.469	0.375	0.281	0.281	0.281	0.500	0.406	0.344	0.281	0.281
28.0	0.500	0.406	0.312	0.281	0.281	0.562	0.469	0.375	0.312	0.312
30.0	0.531	0.438	0.344	0.312	0.312	0.594	0.500	0.406	0.344	0.312
32.0	0.562	0.469	0.375	0.344	0.344	0.625	0.531	0.406	0.375	0.344
34.0	0.625	0.500	0.406	0.344	0.344	0.688	0.562	0.438	0.375	0.344
36.0	0.656	0.531	0.438	0.375	0.375	0.719	0.594	0.469	0.406	0.375
38.0	0.688	0.562	0.469	0.406	0.406	0.750	0.625	0.500	0.438	0.406
40.0	0.719	0.594	0.500	0.406	0.406	0.781	0.688	0.531	0.469	0.438
42.0	0.750	0.656	0.500	0.438	0.438	0.844	0.719	0.562	0.500	0.469

Table 5.2.3c

	MAOP ≤ 700 psi					MAOP ≤ 800 psi				
	SMYS (psi) ≥					SMYS (psi) ≥				
D (in.)	35000	42000	52000	60000	70000	35000	42000	52000	60000	70000
≤ 6.625	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
8.625	0.188	0.188	0.188	0.188	0.188	0.203	0.188	0.188	0.188	0.188
10.75	0.219	0.188	0.188	0.188	0.188	0.250	0.203	0.188	0.188	0.188
12.75	0.281	0.219	0.188	0.188	0.188	0.312	0.250	0.188	0.188	0.188
14.0	0.312	0.250	0.188	0.188	0.188	0.344	0.281	0.219	0.188	0.188
16.0	0.344	0.281	0.219	0.188	0.188	0.375	0.312	0.250	0.219	0.188
18.0	0.375	0.312	0.250	0.219	0.219	0.438	0.344	0.281	0.226	0.219
20.0	0.438	0.344	0.281	0.226	0.226	0.469	0.406	0.312	0.250	0.250
22.0	0.469	0.406	0.312	0.281	0.226	0.500	0.438	0.344	0.281	0.250
24.0	0.500	0.438	0.344	0.281	0.250	0.562	0.469	0.375	0.312	0.281
26.0	0.562	0.469	0.375	0.312	0.281	0.625	0.500	0.406	0.344	0.312
28.0	0.594	0.500	0.406	0.344	0.281	0.656	0.562	0.438	0.375	0.312
30.0	0.656	0.531	0.438	0.375	0.312	0.719	0.594	0.469	0.406	0.344
32.0	0.688	0.562	0.469	0.406	0.344	0.750	0.625	0.500	0.438	0.375
34.0	0.750	0.625	0.500	0.438	0.375	0.812	0.688	0.531	0.469	0.406
36.0	0.781	0.656	0.531	0.469	0.375	0.844	0.719	0.562	0.500	0.438
38.0	0.844	0.688	0.562	0.500	0.406	0.906	0.750	0.625	0.531	0.438
40.0	0.875	0.750	0.594	0.500	0.438	0.938	0.812	0.656	0.562	0.469
42.0	0.938	0.781	0.625	0.531	0.469	1.000	0.844	0.688	0.594	0.500

Table 5.2.3d

Minimum Nominal Wall Thicknesses (in.) for Uncased Carrier Pipe (continued)

	MAOP ≤ 900 psi					MAOP ≤ 1000 psi				
	SMYS (psi) ≥					SMYS (psi) ≥				
D (in.)	35000	42000	52000	60000	70000	35000	42000	52000	60000	70000
≤ 6.625	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
8.625	0.219	0.188	0.188	0.188	0.188	0.250	0.188	0.188	0.188	0.188
10.75	0.279	0.219	0.188	0.188	0.188	0.307	0.250	0.188	0.188	0.188
12.75	0.312	0.281	0.219	0.188	0.188	0.344	0.281	0.250	0.188	0.188
14.0	0.344	0.312	0.250	0.203	0.188	0.375	0.312	0.250	0.219	0.188
16.0	0.406	0.344	0.281	0.219	0.188	0.438	0.375	0.312	0.250	0.219
18.0	0.469	0.375	0.312	0.250	0.219	0.500	0.406	0.344	0.281	0.250
20.0	0.500	0.438	0.344	0.281	0.250	0.562	0.469	0.375	0.312	0.281
22.0	0.562	0.469	0.375	0.312	0.281	0.625	0.500	0.406	0.344	0.281
24.0	0.625	0.500	0.406	0.344	0.281	0.688	0.562	0.438	0.375	0.312
26.0	0.656	0.562	0.438	0.375	0.312	0.750	0.594	0.469	0.406	0.312
28.0	0.719	0.594	0.469	0.406	0.312	0.750	0.656	0.531	0.438	0.344
30.0	0.750	0.625	0.500	0.438	0.344	0.812	0.688	0.562	0.469	0.375
32.0	0.812	0.688	0.562	0.469	0.375	0.875	0.719	0.594	0.531	0.406
34.0	0.875	0.719	0.594	0.500	0.406	0.938	0.781	0.625	0.562	0.438
36.0	0.906	0.781	0.625	0.531	0.438	1.000	0.812	0.688	0.594	0.469
38.0	0.969	0.812	0.656	0.562	0.438	1.062	0.875	0.719	0.625	0.500
40.0	1.031	0.875	0.688	0.625	0.469	1.125	0.906	0.750	0.656	0.531
42.0	1.062	0.906	0.750	0.656	0.500	1.188	0.969	0.781	0.688	0.562

Table 5.2.3e

	MAOP ≤ 1100 psi					MAOP ≤ 1200 psi				
	SMYS (psi) ≥					SMYS (psi) ≥				
D (in.)	35000	42000	52000	60000	70000	35000	42000	52000	60000	70000
≤ 5.563	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
6.625	0.188	0.188	0.188	0.188	0.188	0.203	0.188	0.188	0.188	0.188
8.625	0.250	0.203	0.188	0.188	0.188	0.277	0.219	0.188	0.188	0.188
10.75	0.307	0.250	0.203	0.188	0.188	0.344	0.277	0.219	0.188	0.188
12.75	0.375	0.312	0.250	0.219	0.188	0.406	0.330	0.281	0.226	0.188
14.0	0.406	0.344	0.281	0.226	0.219	0.438	0.375	0.312	0.250	0.219
16.0	0.469	0.406	0.312	0.281	0.219	0.500	0.406	0.344	0.281	0.250
18.0	0.531	0.438	0.344	0.312	0.250	0.562	0.469	0.375	0.344	0.281
20.0	0.594	0.500	0.406	0.344	0.281	0.625	0.531	0.438	0.375	0.312
22.0	0.625	0.531	0.438	0.375	0.312	0.688	0.562	0.469	0.406	0.344
24.0	0.688	0.594	0.469	0.406	0.344	0.750	0.625	0.500	0.438	0.375
26.0	0.750	0.625	0.500	0.438	0.375	0.812	0.688	0.562	0.469	0.406
28.0	0.812	0.688	0.562	0.469	0.406	0.875	0.719	0.594	0.500	0.438
30.0	0.875	0.750	0.594	0.531	0.438	0.938	0.812	0.625	0.562	0.469
32.0	0.938	0.781	0.625	0.562	0.469	1.000	0.875	0.688	0.594	0.500
34.0	1.000	0.844	0.688	0.594	0.500	1.062	0.875	0.719	0.625	0.531
36.0	1.062	0.875	0.719	0.625	0.531	1.125	0.938	0.750	0.656	0.562
38.0	1.125	0.938	0.750	0.656	0.562	1.188	1.000	0.812	0.719	0.594
40.0	1.156	0.969	0.812	0.688	0.594	1.250	1.031	0.844	0.750	0.625
42.0	1.250	1.031	0.844	0.750	0.625	1.312	1.094	0.906	0.781	0.656

Table 5.2.3f

Minimum Nominal Wall Thicknesses (in.) for Uncased Carrier Pipe (continued))

	MAOP ≤ 1300 psi					MAOP ≤ 1400 psi				
	SMYS (psi) ≥					SMYS (psi) ≥				
D (in.)	**35000**	**42000**	**52000**	**60000**	**70000**	**35000**	**42000**	**52000**	**60000**	**70000**
≤ 5.563	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
6.625	0.219	0.188	0.188	0.188	0.188	0.250	0.188	0.188	0.188	0.188
8.625	0.277	0.250	0.188	0.188	0.188	0.312	0.250	0.219	0.188	0.188
10.75	0.344	0.307	0.250	0.203	0.188	0.365	0.307	0.250	0.219	0.219
12.75	0.438	0.344	0.281	0.256	0.219	0.438	0.375	0.312	0.256	0.250
14.0	0.469	0.375	0.312	0.279	0.226	0.500	0.406	0.344	0.281	0.281
16.0	0.531	0.438	0.375	0.312	0.281	0.562	0.469	0.375	0.344	0.312
18.0	0.594	0.500	0.406	0.344	0.312	0.625	0.531	0.438	0.375	0.344
20.0	0.656	0.562	0.438	0.375	0.344	0.688	0.594	0.469	0.406	0.375
22.0	0.719	0.594	0.500	0.438	0.406	0.750	0.656	0.531	0.469	0.375
24.0	0.812	0.656	0.531	0.469	0.406	0.844	0.688	0.562	0.500	0.438
26.0	0.844	0.719	0.594	0.500	0.438	0.906	0.750	0.625	0.531	0.469
28.0	0.906	0.781	0.625	0.531	0.469	0.969	0.812	0.656	0.594	0.500
30.0	0.969	0.812	0.688	0.594	0.500	1.031	0.875	0.719	0.625	0.531
32.0	1.031	0.875	0.719	0.625	0.531	1.094	0.938	0.750	0.656	0.562
34.0	1.125	0.938	0.750	0.656	0.562	1.156	1.000	0.812	0.719	0.594
36.0	1.188	1.000	0.812	0.719	0.625	1.250	1.062	0.875	0.750	0.656
38.0	1.250	1.062	0.844	0.750	0.656	1.312	1.094	0.906	0.781	0.688
40.0	1.312	1.094	0.906	0.781	0.688	1.375	1.156	0.938	0.844	0.719
42.0	1.375	1.156	0.938	0.844	0.719	1.469	1.219	1.000	0.875	0.750

Table 5.2.3g

	MAOP ≤ 1500 psi					MAOP ≤ 1600 psi				
	SMYS (psi) ≥					SMYS (psi) ≥				
D (in.)	**35000**	**42000**	**52000**	**60000**	**70000**	**35000**	**42000**	**52000**	**60000**	**70000**
≤ 4.5	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
5.563	0.219	0.188	0.188	0.188	0.188	0.219	0.188	0.188	0.188	0.188
6.625	0.250	0.203	0.188	0.188	0.188	0.280	0.219	0.188	0.188	0.188
8.625	0.312	0.277	0.219	0.188	0.188	0.344	0.277	0.250	0.219	0.188
10.75	0.406	0.344	0.279	0.226	0.219	0.438	0.344	0.279	0.250	0.219
12.75	0.469	0.406	0.312	0.281	0.250	0.500	0.406	0.344	0.312	0.250
14.0	0.500	0.438	0.344	0.312	0.250	0.562	0.469	0.375	0.312	0.281
16.0	0.594	0.500	0.406	0.344	0.312	0.625	0.531	0.438	0.375	0.312
18.0	0.656	0.562	0.469	0.406	0.344	0.688	0.594	0.469	0.406	0.344
20.0	0.719	0.625	0.494	0.438	0.375	0.781	0.656	0.531	0.469	0.406
22.0	0.812	0.688	0.562	0.469	0.406	0.844	0.719	0.594	0.500	0.438
24.0	0.875	0.750	0.594	0.531	0.438	0.938	0.781	0.625	0.562	0.469
26.0	0.938	0.812	0.656	0.562	0.500	1.000	0.844	0.688	0.594	0.500
28.0	1.031	0.875	0.688	0.625	0.531	1.062	0.906	0.750	0.656	0.562
30.0	1.094	0.938	0.750	0.656	0.562	1.156	0.969	0.781	0.688	0.594
32.0	1.156	0.969	0.812	0.688	0.594	1.219	1.031	0.844	0.719	0.625
34.0	1.250	1.031	0.844	0.750	0.625	1.312	1.094	0.906	0.781	0.656
36.0	1.312	1.094	0.906	0.781	0.688	1.375	1.156	0.938	0.812	0.719
38.0	1.375	1.156	0.938	0.844	0.719	1.469	1.219	1.000	0.875	0.750
40.0	1.438	1.219	1.000	0.875	0.750	1.531	1.281	1.062	0.906	0.781
42.0	1.531	1.281	1.062	0.938	0.781	—	1.344	1.094	0.969	0.844

Table 5.2.3h

Minimum Nominal Wall Thicknesses (in.) for Uncased Carrier Pipe (continued)

	MAOP ≤ 1700 psi					MAOP ≤ 1800 psi				
	SMYS (psi) ≥					SMYS (psi) ≥				
D (in.)	35000	42000	52000	60000	70000	35000	42000	52000	60000	70000
≤ 4.0	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
4.5	0.188	0.188	0.188	0.188	0.188	0.203	0.188	0.188	0.188	0.188
5.563	0.258	0.188	0.188	0.188	0.188	0.258	0.219	0.188	0.188	0.188
6.625	0.280	0.250	0.188	0.188	0.188	0.312	0.250	0.219	0.188	0.188
8.625	0.375	0.312	0.250	0.219	0.188	0.375	0.312	0.250	0.219	0.188
10.75	0.438	0.365	0.312	0.256	0.219	0.469	0.406	0.312	0.279	0.250
12.75	0.531	0.438	0.375	0.312	0.281	0.562	0.469	0.375	0.344	0.281
14.0	0.594	0.500	0.406	0.344	0.312	0.625	0.500	0.406	0.375	0.312
16.0	0.656	0.562	0.438	0.406	0.344	0.688	0.594	0.469	0.406	0.344
18.0	0.750	0.625	0.500	0.438	0.375	0.781	0.656	0.531	0.469	0.406
20.0	0.812	0.688	0.562	0.500	0.406	0.875	0.719	0.594	0.500	0.438
22.0	0.906	0.750	0.625	0.531	0.469	0.969	0.781	0.656	0.562	0.500
24.0	1.000	0.812	0.656	0.594	0.500	1.031	0.875	0.719	0.625	0.531
26.0	1.062	0.906	0.719	0.625	0.531	1.125	0.938	0.750	0.656	0.562
28.0	1.156	0.969	0.781	0.688	0.594	1.219	1.000	0.812	0.719	0.625
30.0	1.219	1.031	0.844	0.719	0.625	1.312	1.094	0.875	0.750	0.656
32.0	1.312	1.094	0.875	0.781	0.656	1.375	1.156	0.938	0.812	0.688
34.0	1.375	1.156	0.938	0.812	0.688	1.500	1.219	1.000	0.875	0.750
36.0	1.469	1.219	1.000	0.875	0.750	1.562	1.312	1.062	0.906	0.781
38.0	1.562	1.312	1.062	0.906	0.781	—	1.375	1.125	0.969	0.844
40.0	—	1.375	1.094	0.969	0.844	—	1.438	1.156	1.000	0.875
42.0	—	1.438	1.156	1.000	0.875	—	1.500	1.219	1.062	0.906

Table 5.2.3i

	MAOP ≤ 1900 psi					MAOP ≤ 2000 psi				
	SMYS (psi) ≥					SMYS (psi) ≥				
D (in.)	35000	42000	52000	60000	70000	35000	42000	52000	60000	70000
≤ 3.5	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188	0.188
4.0	0.188	0.188	0.188	0.188	0.188	0.219	0.188	0.188	0.188	0.188
4.5	0.219	0.188	0.188	0.188	0.188	0.219	0.188	0.188	0.188	0.188
5.563	0.258	0.219	0.188	0.188	0.188	0.281	0.250	0.188	0.188	0.188
6.625	0.312	0.250	0.219	0.188	0.188	0.344	0.280	0.219	0.188	0.188
8.625	0.406	0.344	0.281	0.277	0.219	0.438	0.344	0.281	0.250	0.219
10.75	0.500	0.406	0.344	0.312	0.250	0.531	0.438	0.375	0.312	0.256
12.75	0.594	0.500	0.406	0.344	0.312	0.625	0.531	0.438	0.375	0.312
14.0	0.656	0.531	0.438	0.375	0.344	0.688	0.562	0.469	0.406	0.344
16.0	0.750	0.625	0.500	0.438	0.375	0.781	0.656	0.531	0.469	0.406
18.0	0.812	0.688	0.562	0.500	0.438	0.875	0.719	0.594	0.500	0.438
20.0	0.906	0.781	0.625	0.531	0.469	0.969	0.812	0.656	0.562	0.500
22.0	1.000	0.844	0.688	0.594	0.500	1.062	0.875	0.719	0.625	0.531
24.0	1.094	0.906	0.750	0.656	0.562	1.156	0.969	0.781	0.688	0.594
26.0	1.188	1.000	0.812	0.688	0.594	1.250	1.062	0.844	0.750	0.625
28.0	1.312	1.062	0.875	0.750	0.656	1.344	1.125	0.906	0.781	0.688
30.0	1.375	1.156	0.938	0.812	0.688	1.438	1.219	0.969	0.844	0.719
32.0	1.469	1.219	1.000	0.844	0.750	1.531	1.281	1.031	0.906	0.781
34.0	1.562	1.312	1.062	0.906	0.781	—	1.375	1.094	0.969	0.812
36.0	—	1.375	1.125	0.969	0.844	—	1.438	1.156	1.000	0.875
38.0	—	1.438	1.188	1.031	0.875	—	1.531	1.219	1.062	0.906
40.0	—	1.531	1.219	1.062	0.906	—	—	1.312	1.125	0.969
42.0	—	—	1.281	1.125	0.969	—	—	1.375	1.188	1.000

Table 5.2.3j

5.2.3.1 Allowable Hoop Stress Due to Internal Pressure

The maximum allowable hoop stress due to internal pressure shall be sixty percent of SMYS or per ANSI Code if lower allowable percentage of hoop stress applies.

5.2.3.2 Length of Special Carrier Pipe

Carrier pipe, with nominal wall thicknesses greater than or equal to those shown in Tables 5.2.3 a through j, shall extend from right of way line to right of way line, or 25 ft. from centerline track, whichever distance is greater, unless special conditions exist which prevent this from occurring or as approved by the engineer.

5.2.3.3 Cathodic Protection

Carrier pipes must be coated and cathodically protected to industry standards and test sites for monitoring pipeline provided within 50 ft. of crossing.

Where carrier pipe is cathodically protected, the engineer shall be notified and a suitable test made to ensure that other railway structures and facilities are adequately protected from the cathodic current in accordance with the recommendation of current Reports of Correlating Committee on Cathodic Protection, published by the National Association of Corrosion Engineers.

5.2.4 CONSTRUCTION

5.2.4.1 Special Protection

When the engineer determines there is a possibility of having foreign materials in the subgrade, unusual potential for third party damage exists, or for other reasons, special protection of the carrier pipe will be required. Special protection may require concrete jacketed steel pipe be used, or protection slabs be placed above the pipe, the depth of burial increased, or other means. Soil borings may also be required to determine soil characteristics and to identify if foreign material is present in the bore.

5.2.4.2 Depth of Burial

Carrier line pipe under railway tracks shall not be less than 10 ft. from the base of railway rail to the top of the pipe at its closest point. At all other locations on the rights-of-way the minimum ground cover must be 6 ft. Where it is not possible to secure the above depths, casings as specified in Part 5.1, or other means of protection, will be required.

The Inspection and Testing, and Shutoff Valves specifications are the same as in Article 5.1.

5.2.4.3 Longitudinal Pipelines

Longitudinal pipelines should be located as far as possible from any track. They must not be within 25 ft. of any track and must have a minimum of 6 ft. ground cover over the pipeline up to 50 ft. from centerline of track. Where pipeline is laid more than 50 ft. from centerline of track, minimum cover shall be at least 5 ft. Pipelines must be marked by a sign approved by the engineer every 500 ft. and at every road crossing, streambed, other utility crossing, and at locations of major change in direction of the line. The nominal wall thickness of the pipeline is to be in accordance with Table 5.2.3 a though j.

5.2.4.4 Method of Installation

Installations shall be bored or jacked, and shall have a bored hole diameter essentially the same as the outside diameter of the pipe plus the thickness of the protective coating. If voids should develop or if the bored hole diameter is greater than the outside diameter of the pipe (including coating) by more than approximately 1 in., remedial measures as approved by the engineer shall be taken. Boring operations shall not be stopped if such stoppage would be detrimental to the railway.

5.2.5 Approval of Plans

Plans for proposed installation shall be submitted to and meet the approval of the engineer before construction is begun.

Plans shall be drawn to scale showing the relation of the proposed pipeline to railway tracks, angle of crossing, location of valves, railway survey station, right-of-way lines and general layout of tracks and railway facilities. Plans should also show a cross section (or sections) from field survey, showing pipe in relation to actual profile of ground and tracks. If open-cutting or tunneling is necessary, details of sheeting and method of supporting tracks or driving tunnel shall be shown.

In addition to the above, plans should contain the following data:

	Carrier Pipe
Contents to be handled	....
Outside Diameter	....
Pipe Material	....
Specification and grade	....
Wall thickness	....
Actual Working pressure	....
Type of joint	....
Coating	....
Method of installation	....
Bury: Base of rail to top of carrier	...... ft. ...... in.
Bury: (Not beneath tracks)	...... ft. ...... in.
Bury: (Roadway ditches)	.... ft.
Distance C.L. track to face of jacking/receiving pits	...... ft. ...... in.
Bury: Base of rail to bottom jacking/receiving pits	...... ft. ...... in.
Cathodic protection	yes ...... no ......

5.2.6 Execution of Work

The execution of work on railway rights-of-way, including the supporting of tracks, shall be subject to the inspection and direction of the engineer.

5.2.7 Commentary

A commentary on the "Design of Uncased Pipelines at Railroad Crossings" and the "Guidelines for Pipelines Crossing Railroads" outline the design methodology as developed by Cornell University under the sponsorship of the Gas Research Institute. This information is published in A.R.E.A. Bulletin No. 738 Vol. 93, 1992.

5.3 SPECIFICATIONS FOR PIPELINES CONVEYING NON-FLAMMABLE SUBSTANCES

5.3.1 Scope

Pipelines included under these specifications are those installed to carry steam, water or any non-flammable substance except nonflammable gas products as covered in Article 5.2 which, from its nature or pressure, might cause damage if escaping on or in the vicinity of railway property. The term "engineer" as used herein means chief engineer of the railway company, or his authorized representative.

5.3.2 General Requirements

Pipelines under railway tracks and across railway rights-of-way shall be encased in a larger pipe or conduit called the casing pipe as indicated in Fig.5.3.2.1. Casing pipe may be omitted under the following conditions:

(a) Under secondary or industry tracks as approved by the engineer.
(b) On pipelines in streets where joints are of leakproof construction and the pipe material will safely withstand the combination of internal pressure and external loads.
(c) For non-pressure sewer crossings where the pipe strength is capable of withstanding railway loading.

Pipelines shall be installed under tracks by boring or jacking, if practicable.

Pipelines shall be located, where practicable, to cross tracks at approximately right angles thereto but preferably at not less than 45 degrees and shall not be placed within culverts nor under railway bridges where there is likelihood of restricting the area required for the purposes for which the bridges or culverts were built, or of endangering the foundations.

Pipelines laid longitudinally on railway rights-of-way shall be located as far as practicable from any tracks or other important structures. If located within 25 ft. of the centerline of any track or where there is danger of damage from leakage to any bridge, building or other important structure, the carrier pipe shall be encased or of special design as approved by the engineer.

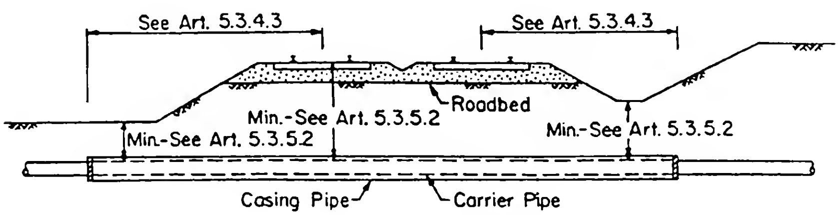


Figure 5.3.2.1

Any replacement of a carrier pipe shall be considered a new installation, subject to the requirements of these specifications.

Where laws or orders of public authority prescribe a higher degree of protection than specified herein, then the higher degree of protection so prescribed shall supersede the applicable portions.

Pipelines and casing pipe shall be suitably insulated from underground conduits carrying electric wires on railway rights-of-way.

5.3.3 Carrier Pipe

Carrier line pipe and joints shall be of acceptable material and construction as approved by the engineer. Joints for carrier line pipe operating under pressure shall be mechanical or welded type.

The pipe shall be laid with sufficient slack so that it is not in tension.

5.3.4 Casing Pipe

Casing pipe and joints shall be leakproof construction, capable of withstanding railroad loading. The inside diameter of the casing pipe shall be at least 2 in. greater than the largest outside diameter of the carrier pipe, joints or couplings, for carrier pipe less than 6 in. in diameter; and at least 4 in. greater for carrier pipe 6 in. and over in diameter. It shall, in all cases, be great enough to allow the carrier pipe to be removed subsequently without disturbing the casing pipe or roadbed.

When casing is installed without benefit of a protective coating or said casing is not cathodically protected, the wall thickness shown above shall be increased to the nearest standard size which is a minimum of 0.063 in. greater than the thickness required except for diameters under 12-3/4 in.

Table 5.3.4.1

Minimum Wall Thickness for Steel Casing Pipe for E80 Loading

Nominal Diameter (inches)	When coated or cathodically protected Nominal Thickness (inches)	When not coated or cathodically protected Nominal Thickness (inches)
12-3/4 and under	0.188	0.188
14	0.188	0.250
16	0.219	0.281
18	0.250	0.312
20 and 22	0.281	0.344
24	0.312	0.375
26	0.344	0.406
28	0.375	0.438
30	0.406	0.469
32	0.438	0.500
34 and 36	0.469	0.531
38	0.500	0.562
40	0.531	0.594
42	0.562	0.625
44 and 46	0.594	0.656
48	0.625	0.688
50	0.656	0.719
52	0.688	0.750
54	0.719	0.781
56 and 58	0.750	0.812
60	0.781	0.844
62	0.812	0.875
64	0.844	0.906
66 and 68	0.875	0.938
70	0.906	0.969
72	0.938	1.000

5.3.4.1 Steel Pipe

Steel pipe shall have a specified minimum yield strength, SMYS, of at least 35,000 psi.

5.3.4.2 Concrete and Corrugated Metal Pipe

For pressures under 100 psi in the carrier pipe, the casing pipe may be reinforced concrete pipe conforming to the AREA Specifications for Reinforced Concrete Culvert Pipe, Part 10, Chapter 8 or coated corrugated metal pipe conforming to the AREA specifications for such pipe, Part 4, this Chapter.

5.3.4.3 Length of Pipe

Casing pipe under tracks and across railway rights-of-way shall extend to the greater of the following distances, measured at right angles to centerline of track. If additional tracks are constructed in the future or the railway determines that the roadbed should be widened, the casing shall be extended or other special design incorporated.

(a) 2 ft. beyond toe of slope.
(b) 3 ft. beyond ditch.
(c) A minimum distance of 25 ft. from center of outside track when end of casing is below ground.

5.3.5 Construction

Casing pipe shall be so constructed as to prevent leakage of any substance from the casing throughout its length except at ends. Casing shall be so installed as to prevent the formation of a waterway under the railway, with an even bearing throughout its length, and shall slope to one end (except for longitudinal occupancy).

Where casing and/or carrier pipe is cathodically protected, the engineer shall be notified and suitable test made to ensure that other railway structures and facilities are adequately protected from the cathodic current in accordance with the recommendations of current Reports of Correlating Committee on Cathodic Protection, published by the National Association of Corrosion Engineers.

5.3.5.1 Method of Installation

(a) Installations by open-trench methods shall comply with Installation of Pipe Culverts, Part 4, this Chapter.

(b) Bored or jacked installations shall have a bored hole diameter essentially the same as the outside diameter of the pipe plus the thickness of the protective coating. If voids should develop or if the bored hole diameter is greater than the outside diameter of the pipe (including coating) by more than approximately 1 in., remedial measures as approved by the chief engineer of the railway company shall be taken. Boring operations shall not be stopped if such stoppage would be detrimental to the railway.

(c) Tunneling operations shall be conducted as approved by the engineer. If voids are caused by the tunneling operations, they shall be filled by pressure grouting or by other approved methods which will provide proper support.

5.3.5.2 Depth of Installation

5.3.5.2.1 Casing Pipe

Casing pipe under railway tracks and across railway rights-of-way shall be not less than 5-1/2 ft. from base of railway rail to top of casing at its closest point, except that under secondary or industry tracks this distance may be 4-1/2 ft. On other portions of rights-of-way where casing is not directly beneath any track, the depth from ground surface or from bottom of ditches to top of casing shall not be less than 3 ft.

5.3.5.2.2 Carrier Pipe

Carrier pipe installed under secondary or industry tracks without benefit of casing shall be not less than 4-1/2 ft. from base of railway rail to top of pipe at its closest point nor less than 3 ft. from ground surface or from bottom of ditches.

5.3.5.3 Shut-Off Valves

Accessible emergency shut-off valves shall be installed within effective distances each side of the railway as mutually agreed to by the engineer and the pipeline company. These valves should be marked with signs for identification. Where pipelines are provided with automatic control stations at locations and within distances approved by the engineer, no additional valves shall be required.

5.3.5.4 Longitudinal Pipelines

Pipeline laid longitudinally on railway rights-of-way 50 ft. or less from center line of track, shall be buried not less than 4 ft. from ground surface to top of pipe. Where pipeline is laid more than 50 ft. from center line of track, minimum cover shall be at least 3 ft.

5.3.6 Approval of Plans

Plans for proposed installation shall be submitted to and meet the approval of the engineer before construction is begun.

Plans shall be drawn to scale showing the relation of the proposed pipeline to railway tracks, angle of crossing, location of valves, railway survey station, right-of-way lines and general layout of tracks and railway facilities. Plans should also show a cross section (or sections) from field survey, showing pipe in relation to actual profile of ground and tracks. If open-cutting or tunneling is necessary, details of sheeting and method of supporting tracks or driving tunnel shall be shown.

In addition to the above, plans should contain the following data:

	Carrier Pipe	Casing Pipe
Contents to be handled	....	....
Outside Diameter	....	....
Pipe Material	....	....
Specification and grade	....	....
Wall thickness	....	....
Actual Working pressure	....	....
Type of joint	....	....
Coating	....	....
Method of installation	....	....
Seals: Both ends	one end	Type ....
Bury: Base of rail to top of casing	.... ft.	.... in.
Bury: (Not beneath tracks)	.... ft.	.... in.
Bury: (Roadway ditches)	.... ft.	.... in.
Type, size and spacing of insulators or supports	....	....
Distance C.L. track to face of jacking/receiving pits	.... ft.	.... in.
Bury: Base of rail to bottom jacking/receiving pits	.... ft.	.... in.
Cathodic protection	yes	no ....

5.3.7 Execution of Work

The execution of the work on railway rights-of-way, including the supporting of tracks, shall be subject to the inspection and direction of the engineer.

Proposed 1993 Manual Revisions To Chapter 3 — Ties and Wood Preservation

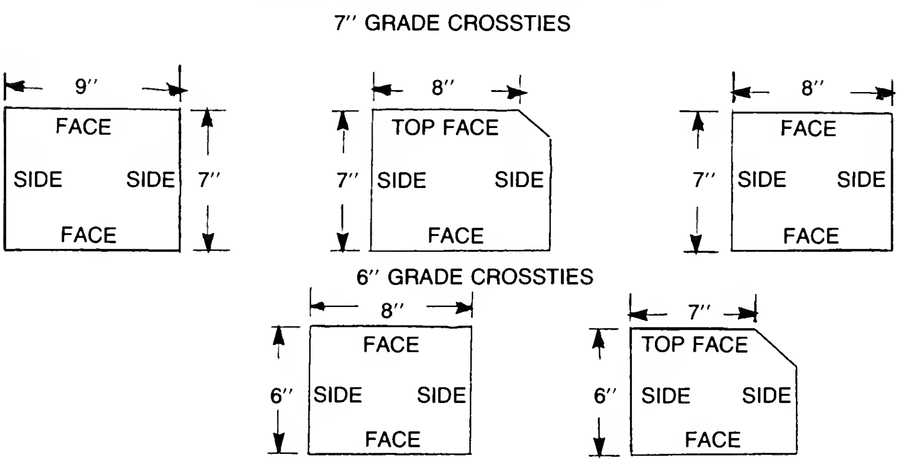
Various revisions to Part 1 — Timber Cross Ties and Part 2 — Timber Switch Ties and a new recommended industrial grade tie specification as Part 12 are proposed. Previously, a similar Part 12 specification was proposed in 1992, but was not approved because it was incomplete. The proposed Part 1 and Part 2 changes and new Part 12 are as follows:

PROPOSED CHANGES TO PART 1 — TIMBER CROSS TIES

1.1 through **1.1.2.2** (Wording remains unchanged)

1.1.3 Design

Size Categories for 7″ & 6″ Crossties
1″ of Wane Allowed — 20% Square 7″ x 8″ Allowed



1.1.3.1 Dimensions (**1.1.3.1** and **1.1.5.9** combined here)

Ties shall be 8-0″, 8′-6″, or 9′-0″ long as specified by the customer. Thickness, width, and length specified are minimum dimensions for green ties. Dry or treated ties may be 1/4″ thinner or narrower than the specified sizes. Ties exceeding these dimensions by more than 1″ shall be rejected. The grade of each tie shall be determined at the point of most wane on the top face of the tie within the rail-bearing areas. The rail-bearing areas are those sections between 20″ and 40″ from the center of the tie. The top of the tie shall be the narrowest face and/or the horizontal face farthest from the heart or pith center.

All rail-bearing areas shall measure as follows: 7″ grade crossties shall be 7″ x 9″ in cross section with a maximum of 1″ of wane in the top rail-bearing areas. A maximum of 20% of the ties in any given quantity may be square-sawn 7″ x 8″ in cross section with *no* wane in the rail-bearing areas. A 6″ grade tie shall be 6″ x 8″ in cross section with a maximum of 1″ of wane permitted in the top rail-bearing areas. For both 6″ and 7″ grade ties, wane shall be permitted on the bottom face so long as it does not exceed 1″ at any given point.

1.1.4 Inspection (Formerly **1.1.5**) (Manufacture moved to **1.1.4.11**)

1.1.4.1 Place (New paragraph number)

Ties shall be inspected at suitable points as specified in the purchase agreement of the railway.

1.1.4.2 Manner (New paragraph number and wording remains unchanged.)

1.1.4.3 Decay (New paragraph number and wording remains unchanged.)

1.1.4.4 Holes (New paragraph number and wording remains unchanged.)

1.1.4.5 Knots (New paragraph number and wording remains unchanged.)

1.1.4.6 Shake (New paragraph number and wording remains unchanged.)

1.1.4.7 Split (New paragraph number)

A split is a separation of the wood extending from one surface to an opposite or adjacent surface. Do not count the end as a surface when measuring the length of a split. (Last sentence added.) (Wording remains unchanged for the rest of the paragraph.)

1.1.4.8 Checks (New paragraph number)

A check is a separation of the wood due to seasoning which appears on one surface only. Do not count the end as a surface. Ties with continuous checks whose depth in a fully seasoned and/or treated tie is greater than 1/4 the thickness *and* longer than 1/2 the length of the tie will be rejected.

1.1.4.9 Slope of Grain (New title and paragraph number)

Except in woods with interlocking grain a slope of grain in excess of 1 in 15 will not be permitted.

1.1.4.10 Bark Seams (New paragraph numbers and wording remains unchanged.)

1.1.4.11 Manufacturing Defects (New title and wording, formerly **1.1.5.8**)

All ties must be straight, square-sawn, cut square at the ends, have top and bottom parallel, and have bark entirely removed. Any ties which do not meet the following characteristics of good manufacture will be rejected:

a. A tie will be considered straight when a straight line from a point on one end to a corresponding point on the other end is no more than 1-1/2″ from the surface at all points.
b. A tie is not well-sawn when its surfaces are cut into with scoremarks more than 1/2″deep, or when its surfaces are not even.
c. The top and bottom of a tie will be considered parallel if any difference at the sides or ends does not exceed 1/8″.
d. For proper seating of nail plates, tie ends must be flat, and will be considered square with a sloped end of up to 1/2″, which equals a 1 in 20 cant.

1.1.5 Delivery (New paragraph numbers)

1.1.5.1 On Railway Premises

Ties shall be delivered and stacked as specified in the purchase agreement of the railway. If ties are to be inspected, they must be placed so that all ties are accessible to the inspector.

1.1.5.2 Risk, Rejection (New paragraph number and wording remains unchanged)

1.1.6 Shipment (New paragraph number and wording remains unchanged.)

1.2 MARKING TIES TO INDICATE SIZE ACCEPTANCE (New paragraph number and wording remains unchanged.)

1.3 EXPLANATION OF CROSS TIE DESIGN (New paragraph number and wording remains unchanged.)

1.4 SPECIFICATIONS FOR MACHINING CROSS TIES (New paragraph number and wording remains unchanged.)

1.4.1 General (New paragraph number)

Delete the word "grooved" from this paragraph.

1.4.2 Adzing (New paragraph number)

Sawn ties provide a flat surface for tie plate seating which precludes the need for adzing. (Delete (a) through (e))

Grooving (Formerly **1.5.3)**

This heading to be deleted because of obsolescence.

1.4.3 Boring (New paragraph number, first two sentences of (a) change as follows:)

(a) Boring for spike holes is optional. If boring is done then boring for spike holes shall conform in size and location to plans provided, with plus or minus 1/16" permitted in each distance between holes. (Remainder of paragraph unchanged)
(b) (Unchanged)
(c) (Unchanged)
(d) Add the following from **1.6.5:**

It is recommended:
(1) That 1/2" holes be bored in hardwood ties for 9/16"cut spikes.
(2) That 9/16" holes be bored in hardwood ties for 5/8" cut spikes.
(3) That 7/16" holes be bored in softwood ties for 9/16" cut spikes.
(4) That 1/2" holes be bored in softwood ties for 5/8" cut spikes.

1.4.4 Trimming (New paragraph number and wording remains unchanged)

1.4.5 Branding (New paragraph number and wording remains unchanged.)

1.5 SIZE OF HOLES BORED FOR SPIKES (Delete existing 1.5, except last 4 sentences moved to new 1.4.3 (d).)

1.6 SPECIFICATIONS FOR TIE PLUGS (New paragraph numbers and wording remains unchanged.)

1.8 SPECIFICATIONS FOR DATING NAILS (Delete for lack of use.)

1.7 SPECIFICATIONS FOR DEVICES TO CONTROL THE SPLITTING OF WOOD CROSS TIES (New numbering for all paragraphs **1.7.1-1.7.4.** Wording remains unchanged.)

1.8 APPLICATION OF ANTI-SPLITTING DEVICES (New number **1.8** instead of **1.10** for all paragraphs. Wording remains unchanged.)

1.11 SPECIFICATIONS FOR TIE COATINGS, THEIR APPLICATIONS AND TESTS (Delete for lack of use.)

PROPOSED CHANGES TO PART 2 — SPECIFICATIONS FOR TIMBER SWITCH TIES

2.1 to 2.1.1 (Wording remains unchanged.)

2.1.1.1 Kinds of Wood Add "Hickories" between Hemlocks and Larches.

2.1.2 to 2.1.2.2 (Wording remains unchanged.)

2.1.3.1 Dimensions (2.1.3.1 and **2.1.5.9** combined here).

All unseasoned or green switch ties shall measure in cross section a minimum of 7″ in side thickness and 9″ in face width. A maximum of 1″ of wane is allowed on the top or bottom faces within the rail-bearing area, which is defined as the section between 12″ from each end of the tie. Seasoned or treated switch ties may be 1/4″ under the specified dimensions for thickness and width, or not more than 1″ over the specified dimensions. Lengths and length tolerances shall be specified by the customer.

All thickness and face width dimensions apply to the rail-bearing area. All determinations of face width shall be made on the top of the switch tie, which is the narrowest horizontal face. If both horizontal faces are of equal width, the top shall be that face with the narrowest or no heartwood.

2.1.4 Inspection (Formerly **2.1.5**) (Manufacture moved to **2.1.4.11**).

2.1.4.1 Place (New paragraph number)

Ties shall be inspected at suitable points as specified in the purchase agreement of the railway.

2.1.4.2 Manner (New paragraph number and wording remains unchanged.)

2.1.4.3 Decay (New paragraph number and wording remains unchanged.)

2.1.4.4 Holes (New paragraph number and wording remains unchanged.)

2.1.4.5 Knots (New paragraph number and wording remains unchanged.)

2.1.4.6 Shake (New paragraph number and wording remains unchanged, but add a sentence:)

The procedure and diagrams shown in 1.1.4.6 for crossties shall also apply to switch ties for measuring the length of a shake.

2.1.4.7 Splits (New paragraph number and delete all old wording. New wording as follows:)

A split is a separation of the wood extending from one surface to an opposite or adjacent surface. Do not count the end as a surface when measuring the length of a split.

In unseasoned or green switch ties, a split no more than 1/8″ wide and/or 5″ long is acceptable. In a seasoned or treated switch tie, a split no more than 1/4″ wide and/or longer than the width of the face across which it occurs is acceptable. A split exceeding the limit is acceptable, provided split limitations and anti-splitting devices are approved by the buyer and properly applied.

2.1.4.8 Checks (New paragraph number. Replaces old **2.1.5.11.**)

A check is a separation of the wood due to seasoning which appears on one surface only. Do not count the end as a surface when measuring the length of a check. Ties with continuous checks whose depth in a fully seasoned and/or treated tie is greater than 1/4 the thickness *and* longer than 1/2 the length of the tie will be rejected.

2.1.4.9 Slope of Grain (New title to replace "Slanting Grain" and paragraph number plus wording change:)

Except in woods with interlocking grain, a slope of grain in excess of 1 in 15 will not be permitted.

2.1.4.10 Bark Seams (New paragraph number. Replaces old 2.1.5.10. Wording remains unchanged).

2.1.4.11 Manufacturing Defects (New paragraph number, title and wording, formerly **2.1.5.8**).

All ties must be straight, square-sawn, cut square at the ends, have top and bottom parallel, and have bark entirely removed. Any ties which do not meet the following characteristics of good manufacture will be rejected:

a. A tie will be considered straight when a straight line from a point on one end to a corresponding point on the other end is no more than 2″ from the surface at all points.
b. A tie is not well-sawn when its surfaces are cut into with scoremarks more than 1/2″ deep, or when its surfaces are not even.

c. The top and bottom of a tie will be considered parallel if any difference at the sides or ends does not exceed 1/4".
d. For proper seating of nail plates, tie ends must be flat, and will be considered square with a sloped end of up to 1/2", which equals a 1 in 20 cant.

2.1.5 Delivery (New paragraph number).

2.1.5.1 On Railway Premises (New paragraph number and wording change. Delete all former wording and replace with:)

Ties shall be delivered and stacked as specified in the purchase agreement of the railway. If ties are to be inspected, they must be placed so that all ties are accessible to the inspector.

2.1.5.2 Risk, Rejection (New paragraph number and wording remains unchanged.)

2.1.5.3 Species Groups for Seasoning and Treating (New title and paragraph number. Replaces **2.1.6.3 Grouping** and **2.1.6.4 Class T.** All wording changed as follows:)

Switch ties shall be grouped as shown below for air-seasoning or artificial seasoning and subsequent preservative treatment. Only the kinds of wood named in a group may be processed together. (The four columns of groups remain unchanged with one exception–add "Hickories" to Group Td).

2.1.6 Shipment (New paragraph number and wording stays the same).

Part 12

Specifications for Timber Industrial Grade Cross Ties

12.1 SPECIFICATIONS FOR TIMBER INDUSTRIAL GRADE CROSS TIES

12.1.1 Material

12.1.1.1 Kinds of Wood

Before manufacturing ties, producers shall ascertain which of the following kinds of wood suitable for cross ties will be accepted:

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

12.1.2 General

All procedures regarding quality, manufacture, inspection, shipment, and delivery will comply fully with those specified for grade cross ties in Part 1 unless excepted by Part 12.

12.1.3 Classification and Design

The following sizes, lengths, minimum faces and tolerances are allowed:

Grade	Dimensions	Minimum Faces Allowed
6″ IG	6″ x 8″ x 8′0″/8′6″	6″ face on top or bottom
7″ IG	7″ x 8″ x 8′0″/8′6″	6″ face on top or bottom
7″ IG	7″ x 9″ x 8′0″/8′6″	6″ face on top or bottom

The above minimum face requirements apply to the rail-bearing areas, which are the areas between 20″ and 40″ from the middle of the industrial grade crosstie. Outside the rail-bearing areas, wane will be limited to half the face width on the top or bottom of the tie. The grade of each tie shall be determined at the point of most wane, on the top or bottom, within the rail-bearing areas. (The top is defined as the horizontal face farthest from the heartwood or pith center).

Dry or treated ties may be 1″ narrower or 1/2″ thinner than the specified sizes. Thickness and width may not vary more than 1″ from end to end. The tie body may be out of square by no more than 1″ throughout the length. Tie length may vary from + 1″ to – 3″ for the length specified.

12.1.4 Definitions of Defects

12.1.4.1 Wane

Wane is defined as bark or the lack of wood (see 12.1.3 for allowance).

12.1.4.2 Decay

A decayed knot greater than 3/4″ in diameter will be rejected within the rail-bearing area. Also, slight incipient decay may be allowed if the tie, as a whole, is basically of good quality. Decay is allowed outside the rail-bearing area if the decayed area does not exceed 2″ in diameter. Ties with decay up to 2″ in diameter appearing in *both* ends of the tie will be rejected.

12.1.4.3 Holes

Ties having holes on any surface within the rail-bearing areas that are greater than 1/2″ in diameter or greater than 3″ deep will be rejected. Holes on any surface outside the rail-bearing areas which are greater than 3″ in diameter or deeper than 4″ will be rejected.

12.1.4.4 Knots

A knot greater than 3″ in diameter within the rail-bearing area will not be permitted.

12.1.4.5 Shakes

Seasoned or treated ties with shakes having a length on the cross-section greater than 5″ or extending to within 1″ of any surface shall be rejected. Length measurements shall be made using 1.1.4.6 as a guide.

12.1.4.6 Splits

A split is a separation of wood extending from one surface to an opposite or adjacent surface—not counting the end as a surface. A seasoned or treated tie with a split greater than 1/2″ wide or 11″ long will be rejected with or without a nail plate.

12.1.4.7 Checks

A check is a separation of wood due to seasoning which appears on one surface only—not counting the end as a surface. Season checks greater than 2″ deep or 3/4″ wide shall be rejected as industrial grade ties.

12.1.4.8 Cross or Spiral Grain

Except in species with interlocking grain, ties having cross, slant, or spiral grain greater than 2″ in 15″ of length will be rejected.

12.1.4.9 Bark Seams

Bark seams will not be acceptable if more than 2″ deep or more than 10″ long anywhere in the tie.

12.1.4.10 Manufacturing Defects

All ties must be straight and have top and bottom parallel. Any ties which do not meet the following characteristics of good manufacture will be rejected:

(a) A tie will be considered straight when a straight line from a point on one end to a corresponding point on the other end is no more than 2″ from the surface at all points.
(b) The top and bottom of a tie will be considered parallel if any difference at the sides or ends does not exceed 1″.
(c) A tie is not well-sawn when its surfaces are cut with scoremarks more than 1″ deep.
(d) For proper seating of nail plates, tie ends must be flat, and will be considered square with a sloped end of up to 1/2″, which equals a 1 in 20 cant.

Proposed 1993 Manual Revisions To Chapter 4 — Rail

Changes proposed to Chapter 4 are in Part 2 — Specifications and Part 3 — Report Forms. In Part 2 revisions involve Sections 1 through 4 in updating the rail specification due to previously eliminated 90 and 100 lb./yd. rail sections, and proposed increase of the minimum hardness of standard rail to 300 HB. Also for Part 2, Section 14 on Acceptance has been expanded with the addition of Article 14.3 and revision of Appendix 1, Section 6, making obsolete Part 3 Forms 401-A through 401-D. A new microalloyed joint bar has been added to Part 2's Quenched Carbon-Steel Joint Bar and Forged Compromise Joint Bar Specification on page 4-2-12. Specifically the changes proposed are as follows:

PART 2 — SPECIFICATIONS

Proposed new Sections 1. through 4.:

1. Scope

1.1 These specifications cover steel tee rails weighing 115 lb./yd. and over for use in railway track.

1.2 Drawings of recommended rail sections are shown on pages 4-1-3 through 4-1-6.1 of this manual.

1.3 ASTM Specifications A 1, A 2, and A 759 are referenced for tee rails weighing 60 lb./yd. and over, girder rails, and crane rails, respectively.

1.4 Supplementary requirements S1 and S2 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.

2.4 Rails shall be furnished in the as-rolled (standard and alloy), head-hardened, fully-heat-treated, or on-line-hardened conditions as agreed by purchaser and manufacturer.

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		Product Analysis, Weight Percent Allowance Beyond Limits of Specified Chemical Analysis	
Element	**Min.**	**Max.**	**Under Min.**	**Over Max.**
Carbon	0.72	0.82	0.04	0.04
Manganese	0.80	1.10*	0.06	0.06
Phosphorus	—	0.035	—	0.008
Sulfur	—	0.037	—	0.008
Silicon	0.10	0.50	0.02	0.02***
Nickel	—	0.25**		
Chromium	—	0.25**		
Molybdenum	—	0.10**		
Vanadium	—	0.03**		

*May be extended to 1.25% by the manufacturers to meet the hardness specifications. However, Ni, Cr, Mo, and V contents are restricted.

**Applies only when MN > 1.10%.

***0.05% for continuously cast steel.

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 3.1.

3.2 The chemical composition limits of alloy high-strength rail is subject to agreeement 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 within the following limits:

	Brinell Hardness, HB	
	Minimum	Maximum
Standard Rail	300	—
High-Strength Rail (alloy and heat treated)	341	388*

*May be exceeded provided a fully pearlitic microstructure is maintained.

4.2 The 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 or heat-treatment lot. A test report shall be 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 ASTM E 10, "Standard Test Method for Brinell Hardness of Metallic Materials," latest version.

4.3 If any hardness test result fails to meet the specifications, two additional checks shall be made on the same piece. If both checks meet the specified hardness, the heat or heat treatment lot meets the hardness requirement. If either of the additional checks fails, two further rails in the heat or lot shall be checked. Both of these checks must be satisfactory for the heat or lot to be accepted. If any one of these two checks fails, individual rails may be tested for acceptance.

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

4.5 The method of testing, traverse location and interval, number of rails to be tested, and acceptance criteria for the internal hardness of heat-treated rail shall be subject to agreement of the purchaser and manufacturer.

Proposed new Article 14.3:

14.3 Rails accepted shall be shipped and invoiced based on the calculated weight per yard for the rail section. (*Add the following.*) The rail producer shall furnish to the purchaser the following records of inspection and shipment by the method and in the form agreed upon between the purchaser and the producer.

(a) The chemical analysis of the rails shipped, listed by cast or ingot and heat number, and the specified chemical analysis elements. (See Section 3–Chemical Composition.)

(b) The Brinell hardness of the rails shipped by cast or ingot and heat numbers, and the hardness pattern for hardened rails as agreed upon by purchaser and manufacturer. (See Section 4–Hardness Properties.)

(c) The method of hydrogen elimination.

(d) A shipping statement of the rails shipped which will include the number of pieces of each length, and the total tons shipped in each vehicle (rail car or ship).

(e) A statement that all rails supplied meet the ultrasonic requirements. (See Section 8–Ultrasonic testing).

(f) A statement that all macroetched samples representing the rails supplied meet the macroetch requirements. (See Section 9–Interior Condition/Macroetch)

Proposed replacement wording for Appendix 1, Section 6:

6. The manufacturer shall maintain a complete record of the process for each container of rails.

Proposed new Joint Bar Specification:

SPECIFICATIONS FOR QUENCHED CARBON-STEEL JOINT BARS, MICROALLOYED JOINT BARS AND FORGED COMPROMISE JOINT BARS

1. Scope

1.1 These specifications cover heat treated carbon steel joint bars, microalloyed joint bars, and forged compromise joint bars for general use in standard railroad tracks.

2. Manufacture

2.1 Melting Practice—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 ingots, or by other methods agreed upon by the purchaser and the manufacturer.

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

2.4 Heating and Quenching—Quenched carbon—steel joint bars and forged compromise joint bars shall be uniformly heated for punching, slotting, shaping and forging, and subsequently quenched. Maximum depth of decarburized layer of forged bars shall not exceed 0.040 inches.

2.5 Microalloyed joint bars shall be produced from hot rolled steel sections. Bars shall be sheared or sawed cold, and holes shall be drilled. No reheating and quenching is required.

3. Chemical Composition

3.1 Composition

3.1.1 The chemical composition of the quenched carbon-steel joint bars and forged compromise joint bars, determined as prescribed in 3.2.1, shall be within the limits shown in Table 1, Chemical Analysis.

3.1.2 Finished material representing the heat may be product tested. The product analysis shall be within the limits for product analyses specified in Table 1.

3.1.2 The chemical composition of the microalloyed joint bars shall be agreed upon by the purchaser and the manufacturer. Microalloying shall be accomplished with columbium, vanadium, and nitrogen, or combinations thereof.

3.2 Heat or Cast Analysis

3.2.1 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 the pouring of the heat. Determinations may be made chemically or spectrographically. Any portion of the heat meeting the chemical analysis requirements of Table 1 may be applied. Additionally, any material meeting the product analysis limits shown in Table 1 may be applied against the customer's order after testing such material.

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

3.2.3 Upon request by the purchaser, samples shall be furnished to verify the analysis as determined in 3.2.1.

Table 1. Chemical Composition

Element	Chemical Analysis Weight Percent	Product Analysis Weight Percent Allowance Beyond Limits of Specified Chemical Analysis	
		Under Minimum Limit	Over Minimum Limit
Carbon	0.35 to 0.60	0.040	0.040
Manganese	1.20 max	0.060	0.060
Phosphorus	0.040 max	N/A	0.008
Sulfur	0.050 max	N/A	0.008

4. Tensile Properites

4.1 The material shall conform to the following requirements as to tensile properties:

Tensile strength, min. psi 100,000
Yield point, min. psi 70,000
Elongation in 2 in., min. percent 12
Reduction of area, min. percent 25

4.2 The yield point shall be determined by the drop of the beam or halt in the gage of the testing machine operated at a cross-head speed not to exceed 1/8 in. per min. The tensile strength shall be determined at a speed of head not to exceed 1-1/2 in. per min.

5. Bending Properties

5.1 Bend Test—The bend test specimen specified in Article 6 shall stand being bent cold through 90 deg. without cracking on the outside of the bent portion around a pin the diameter of which is not greater than three times the thickness of the specimen.

5.2 Optional Bend Test—If preferred by the manufacturer and approved by the purchaser, the following bend test may be substituted for that described in paragraph 5.1: A piece of the finished bar shall stand being bent cold through 45 deg. without cracking on the outside of the bent portion around a pin the diameter of which is not greater than three times the greatest thickness of the section.

6. Test Specimens

6.1 Tension and bend test specimens shall be taken from the middle of the head at the center of the finished bars. Tension test specimens shall be machined to the form and dimensions shown in Fig. 1. Bend test specimens may be 1/2 in. square in section or rectangular in section with two parallel faces as rolled and with corners rounded to a radius not over 1/16 in.

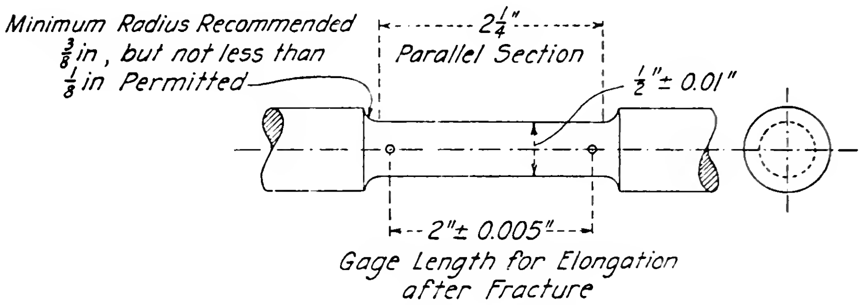


Figure 1. Standard round tension test specimen with 2-in. gage length.

NOTE: The gage length, parallel section, and fillets shall be as shown, but the ends may be of any shape to fit the holders of the testing machine in such a way that the load shall be axial.

7. Number of Tests

7.1 One tension test and one bend test shall be made from each lot of 1,000 bars or fraction thereof, but not less than one test for each heat on each day on which quenched carbon steel bars are heated and quenched, or on which microalloyed joint bars are sheared or sawed.

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

7.3 If the percentage of elongation of any tension test specimen is less than specified in Art. 4 and any part of the fracture is more than 3/4 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 Art. 8.

8. Retests

8.1 If any tensile property of any tension test specimen is less than that specified, and any part of the fracture is outside the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

8.2 If the results of an original tension specimen fail to meet the specified minimum requirements and are within 2000 psi of the required tensile strength, within 1000 psi of the required yield point, or within two percentage units of the required elongation, a retest shall be permitted on two random specimens for each original tension specimen failure from the lot. If all results of these retest specimens meet the specified requirements, the lot shall be accepted.

8.3 If a bend test fails for reasons other than mechanical reasons or flaws in the specimen as described in 8.4 and 8.5, a retest shall be permitted on two random specimens from the same lot. If the results of both test specimens meet the specified requirements, the lot shall be accepted. The retest shall be performed on test specimens that are at air temperature but not less than 60°F.

8.4 If any test specimen fails because of mechanical reasons such as failure of testing equipment or improper specimen preparation, it may be discarded and another specimen taken.

8.5 If any test specimen develops flaws, it may be discarded and another specimen of the same size bar from the same lot substituted.

8.6 For quenched joint bars—If the results of the mechanical tests of any test lot (retests included) do not conform to the requirements specified, the manufacturer may retreat such lot not more than twice, in which case two additional tension tests and two additional bend tests shall be made from such lot, all of which shall conform to the requirements specified.

9. Workmanship

9.1 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 or drilling, 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.

10. Finish

10.1 The material shall be free from injurious defects and shall have a workmanlike finish.

11. Marking and Stamping

11.1 The name or brand of the manufacturer, the section designation, and the year of the manufacture shall be either hot stamped on the side of each of the bars or rolled in raised letters and figures on the side of each of the bars.

11.2 For quenched bars, a serial number representing the heat shall be hot stamped on the outside of the web of each bar, near one end.

11.3 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 the rail the bar is to be used. (If the compromise joint bars are interchangeable, the words gage and out will be omitted.)

12. Inspection

12.1 The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, without charge, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analyses) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

13. Rejection

13.1 Material failing to meet the requirements of these specifications will be rejected.

13.2 Unless otherwise specified, any rejection based on tests made in accordance with Art. 3.1.2 shall be reported to the manufacturer within five working days from the receipt of samples by the purchaser.

13.3 Material that shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

14. Rehearing

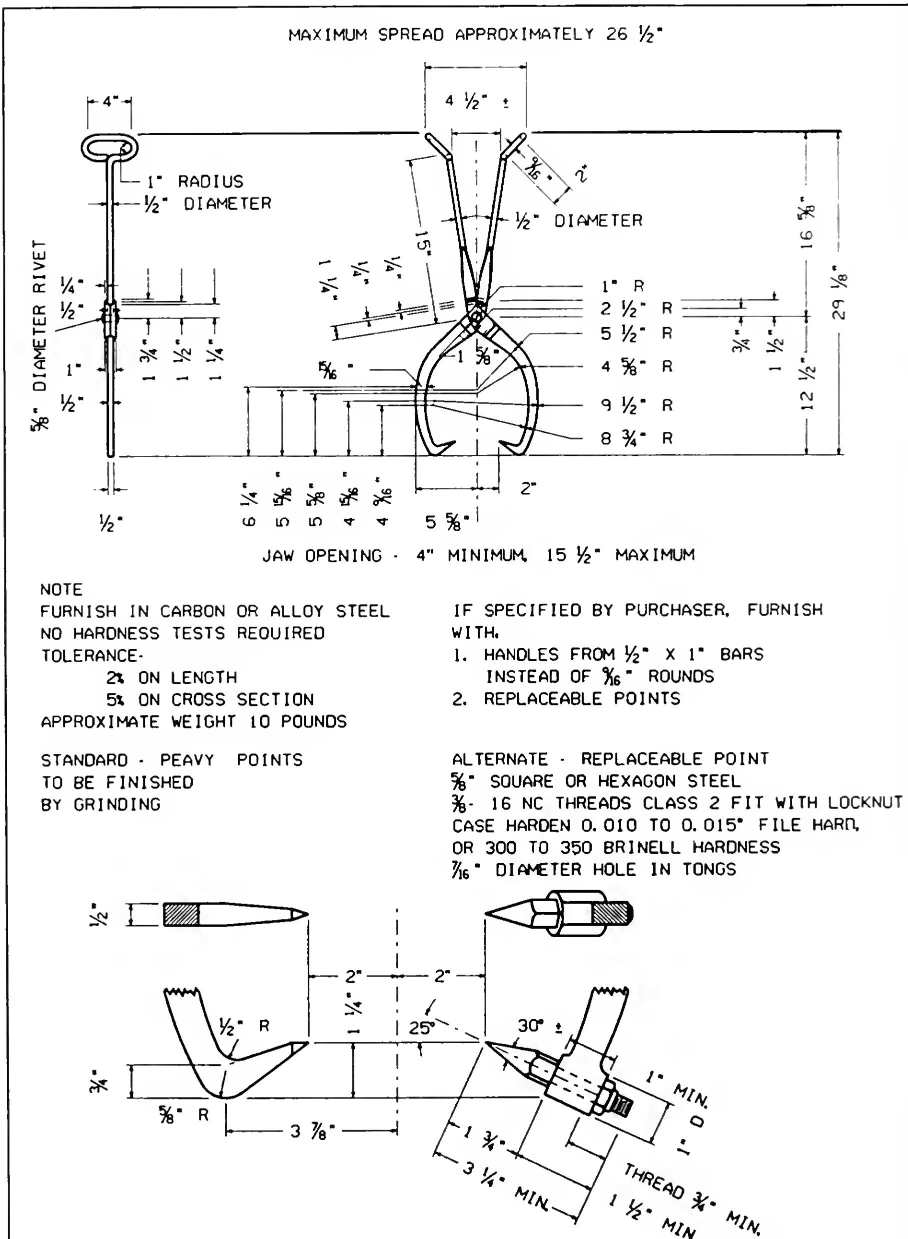
14.1 Samples tested in accordance with Art. 3.1.2 that represent rejected material shall be preserved for two weeks from the date of the test report. In case of dissatisfaction with the results of the tests, the manufacturer may request a rehearing within that time.

PART 3 — REPORT FORMS

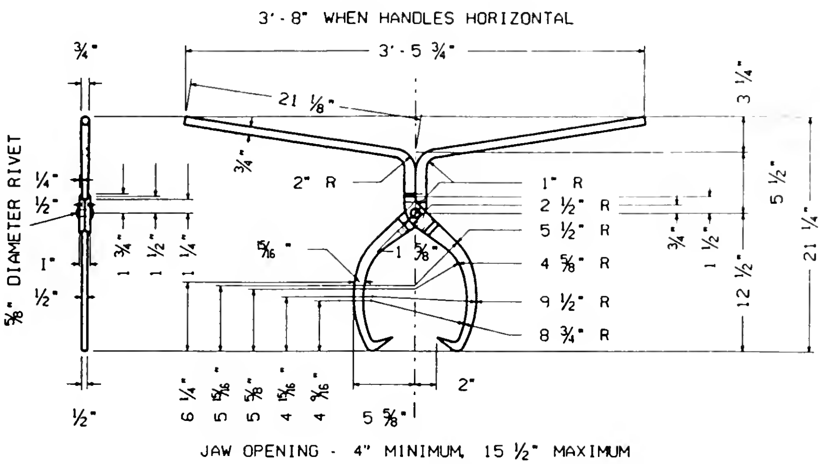
It is proposed to delete reporting forms 401-A, B, C and D in Part 3 that were used for reporting results of Mill Inspection and Shipments. Coverage of this information will be by new requirements in Article 14.3 and Appendix 1, Section 6 that have been proposed herein.

Proposed 1993 Manual Revisions To Chapter 5 — Track

Two revised plans for Part 6 — Specifications and Plans for Track Tools are proposed. Plans 7-93 and 8-93 are revisions of current Plans 7-62 and 8-62, to include replaceable points for one and two man tie tongs as detailed in the following plans:



PLAN 7-93 — AREA TIE TONGS

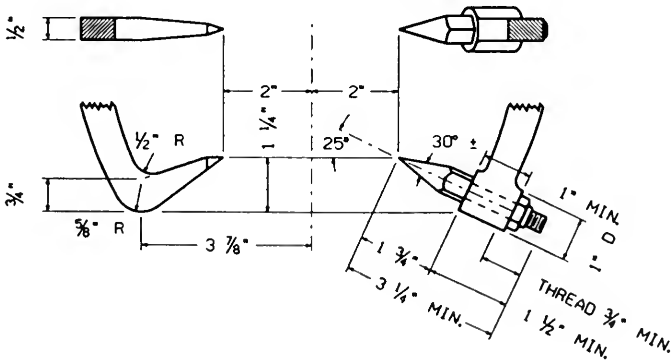


NOTE
FURNISH IN CARBON OR ALLOY STEEL
NO HARDNESS TESTS REQUIRED
TOLERANCE-
2% ON LENGTH
5% ON CROSS SECTION
APPROXIMATE WEIGHT 12.2 POUNDS

IF SPECIFIED BY PURCHASER, FURNISH WITH REPLACEABLE POINTS

STANDARD - PEAVY POINTS
TO BE FINISHED
BY GRINDING

ALTERNATE - REPLACEABLE POINT
⅝" SQUARE OR HEXAGON STEEL
⅜- 16 NC THREADS CLASS 2 FIT WITH LOCKNUT
CASE HARDEN 0.010 TO 0.015" FILE HARD,
OR 300 TO 350 BRINELL HARDNESS
⁷⁄₁₆" DIAMETER HOLE IN TONGS



PLAN 8-93 — AREA TIMBER TONGS

Proposed 1993 Manual Revisions To Chapter 8 — Concrete Structures and Foundations

Total rewrites of Part 1 — Materials, Tests and Construction Requirements and Part 20 — Flexible Sheet Pile Bulkeads are proposed. In Part 1 revisions include addition of new definitions, more stringent specifications on repair or epoxy rebar coatings, reference to ASTM A780 for repair of hot-dip galvanized rebar and numerous editorial alterations. For Part 20 a general rewrite was made without any substantial changes to technical content. Copies of the new Part 1 and 20 may be obtained by writing AREA Headquarters and enclosing $2.00 for each copy.

Proposed 1993 Manual Revisions To Chapter 10 — Concrete Ties

Changes proposed for Chapter 10 are: Replacement of Articles 1.2.1, 1.2.2, 1.2.4 and 1.2.5 with upgraded material to more accurately reflect current practices; The Duggan test will be included as an Appendix referred to in Article 1.2.2; deletion of paragraphs 1.3.2.13(b) and 1.11.6(b) which are no longer requirements needed for the recommended practice; and addition of a new Section 1.11 on concrete tie turnouts, which will necessitate renumbering of current Sections 1.11 to 1.13. The proposed new and replacement material is as follows:

1.2.1 General

Utmost consideration shall be given to concrete properties affecting durability. These include alkali-aggregate reaction, air entrainment and other admixtures, sulphate and other chemical reactions, and concrete tie manufacturing methods.

1.2.2 Concrete

The minimum 28-day-design compressive strength of concrete used for concrete ties shall be 7000 psi as determined by ASTM Method of Test C39. The test cylinders shall be made and stored as specified in ASTM Specification C31.

1.2.2.1 Cement

Cement shall meet the requirements of ASTM Specification C150. It is recommended that cement alkali content of Na_2O equivalent ($Na_2O + 0.658\ K_2O$) be as low as possible and not greater than 0.6%. False set penetration when tested in accordance with ASTM C359 shall be not less than 50 mm initially, 35 mm at intermediate times, and 40 mm after remix.

Alternatively, instead of using low alkali cement to minimize the risk of alkali-aggregate reactivity, pozzolanic materials such as fly ash, silica fume, or slag may be used, provided concretes made from the proposed cement, aggregrates, and pozzolan have a demonstrable and proven durability record. As concrete durability problems may not become evident for some time, it is recommended that a minimum service record of 10 years be used to assess performance.

Cement mill certificates should be obtained on a regular basis during tie production in order to ensure consistency in chemical ingredients. Under no circumstances shall substitution of cement be permitted unless it has been pre-qualified through the tests listed in this section.

1.2.2.2 Aggregates

Both fine and coarse aggregates shall meet the requirements of the AREA Specifications for Aggregates, Part 1, Section 1.3, Chapter 8 of the AREA Manual.

In addition, for preliminary screening, a field review of aggregate performance in existing concrete structures should be conducted, preferably by an experienced petrographer, to determine the historical durability record. Petrographic analysis according to ASTM C295 shall be conducted on each new aggregate source, including new faces or strata in existing pits/quarries, to determine potentially reactive mineral constituents. Analysis shall be repeated at six-month intervals. It may also be desirable to retain the services of a professional geologist.

It is recommended that the Oberholster test according to ASTM C-9 Proposal P214 Proposed Test Method for Accelerated Detection of Potentially Deleterious Expansion of Mortar Bars Due to Alkali-Silica Reaction be conducted on both the fine and coarse aggregates. This Proposed Test is still under ASTM review and will probably be modified before it becomes an ASTM Standard. In the meantime, this test should be carried out with a low alkali cement (less than 0.30% alkali content) and it is recommended that to ensure a job cement is chosen with the level of alkali sufficiently low to avoid deleterious expansion 3 parallel tests are carried out with different levels of sodium hydroxide solution normality: this will allow the expansion against 3 equivalent cement alkali contents to be plotted, thus defining the level of alkali content below which the 0.10% acceptable expansion will not occur. The test as written uses a one normal sodium hydroxide solution, which would be equivalent to about 1.4% equivalent sodium oxide in the cement when used in water/cement ratios of 0.50%. The levels of cement alkali contents available in any area under investigation should be included in the test by adjustment of the normality of the sodium hydroxide solution i.e. a 0.70% normal solution is equivalent to 1.0% cement alkali content, 0.56% normal solution equivalent to 0.8% cement alkali content, etc. Should additional information be desired to confirm findings, the ASTM C227 Mortar Bar Test may be conducted. Tests should be conducted at six month intervals during tie production.

1.2.2.3 Mixing Water

Mixing water shall meet the requirements of the AREA Specifications for Mixing Water, Part 1, Section 1.4, Chapter 8 of the AREA Manual. In addition, the mixing water, including that portion of the mixing water contributed in the form of free moisture on the aggregates, shall not contain deleterious amounts of chloride ion.[1]

1.2.2.4 Admixtures

Chemical admixtures for concrete shall conform with ASTM C494. Additives containing chlorides shall not be used. Where ties will be exposed to freeze-thaw conditions, an air entraining agent according to ASTM C260 shall be used. As a guide, freeze-thaw durability can generally be obtained with 4.5% minimum air in the wet concrete, 3.5% minimum air void content in the hardened concrete, and an air-void spacing factor not exceeding 0.008 inches.

1.2.2.5 Curing

It is recommended that the concrete be cured by a method or procedure such as set forth in Part 1, Section 1.17, Chapter 8 of AREA Manual. During the preset period, the concrete temperature shall not exceed 90°F during the first 3 hours and 105°F during the first 4 hours. With accelerated heat curing, the heating rate shall not exceed 35°F per hour, and the curing temperature within the concrete shall not exceed 140°F, unless the Supplier can prove that the materials used would be satisfactory, in which case temperatures up to 160°F may be used. The recommended maximum temperatures have been taken from European literature, and it is not known at this time if they are directly applicable to North American concretes.

[1] A chloride ion content greater than 400 ppm might be considered detrimental, and it is recommended that levels well below this value be maintained if practicable.

1.2.2.6 Cured Concrete

It is recommended that the complete job mix hardened concrete having proper proportions of aggregates, cement, water and admixtures, and having been cured in the manner intended for ties, be subjected to the following:

a) Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing, ASTM C666, Procedure A (where freeze-thaw environment exists).
The concrete shall have a durability factor of not less than 90%.

b) The Duggan Test Method for Detection of Potentially Deleterious Expansion of Concrete (see Appendix I for description of the Duggan test procedure).

A rigid expansion limit of 0.05% at the 20 day measurement period (relative to the initially saturated concrete) has been recommended. However, this expansion limit has been developed with test cores taken from existing concrete bridge abutments, and it is not known at this time if it is directly applicable to concrete ties. Until more definitive data is developed, the maximum acceptable expansion may be taken as 0.15%.

The above two tests shall be repeated at 6 month intervals.

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 material that are compatible 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 tests specified in Section 1.9.

The tie pad shall have a minimum width equal to the base width of the rail, + 0 and − 1/8 inches. It shall be shaped or have indicators that will provide correct orientation during installation. The thickness of the pad shall not be less than 5 mm.

All pads shall be marked in a permanent manner to indicate manufacturer and pad identification.

1.2.4.2 Material Tests

The following suggested tests for elastomeric pads shall be conducted by the vendor from a batch of material used to manufacture the tie pad specimens for qualification testing. Test specimens must be cured in the same manner as the final product. Each specified test shall be conducted and reported on for three samples. All testing shall be conducted by a laboratory approved by the engineer.

These tests have been selected for their suitability and use in laboratory test evaluations of rubber, plastics and composites or laminates of these materials for concrete tie pads. The tests shall be applied to the individual components of laminated pads.

An ASTM comment that appears throughout these test methods is that correlation between the results from these tests and actual service performance is neither given nor implied, because of the wide variations in service conditions. However, many of these test methods are useful in quality control and, in some cases, product specifications.

(a) Compression set at 23° C (ASTM D 395)
high temp. 70° C (ASTM D 395)
low temp. 20° C (ASTM D 1229)
Method B @ 22 hours & constant deflection

(b) Tensile strength and elongation before and after heat aging. (ASTM D 412 and ASTM D 573). Heat aging shall be two days @ 160°F.

(c) Hardness (ASTM D 2240 –A scale)
(d) Resistance to fluids such as water, acids, alkali, petroleum oils, and synthetic lubricants (ASTM D 471). note: eliminate resistance to ozone.
(e) Vicat softening temperature (ASTM D 1525).
(f) D-C Resistance or conductance of insulating material (ASTM D 257)
(g) Tear resistance (ASTM D 624)
(h) Abrasion resistance (ASTM D 2228 or D 1242).
(i) Rubber properties in compression and shear (Mechanical oscillograph), (ASTM D 945).

1.2.4.3 Tie Pad Tests

(No changes)

1.2.5 Insulation

Insulation shall be used where necessary to prevent interference with the 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.

1.2.5.1 Material Tests

For elastomeric insulators, the supplier shall submit results of industry standard tests covering the following properties.

(a) Electrical resistivity (ASTM D 257)
(b) Tensile and elongation (ASTM D 638)
(c) Notched Izod Impact (ASTM D 256)
(d) Heat Deflection Temp. (ASTM D 648)
(e) Flexural Modulus (ASTM D 790)
(f) Flexural strength (ASTM D 790)
(g) Rockwell hardness, R scale
(h) Water Absorbtion (ASTM D 570)
(i) Resistance to ozone (ASTM D 518)

1.11 TIES FOR TURNOUTS

1.11.1 General

Concrete ties used in turnouts are subjected to loadings and ballast support conditions considerably different from cross ties in standard track. The trackwork mounted on the ties, and the passage of rail traffic, especially on the turnout track, generate non-uniform dynamic loads which must be considered in the tie design.

The dimensions, and tolerances for manufacturing turnout ties are no more critical than standard track ties, but the calculations are much more complex because they must be done precisely for each individual tie in the turnout.

The specific provisions below for turnout ties are to be followed in addition to the regular requirements of Chapter 10. Where no specific provisions are stated, the general requirements for concrete cross ties are to be used.

1.11.2 Layout

1.11.2.1 Tie Orientation

Ties may be oriented at right angles to the straight track, per Figure XII (a) in which case the ties with cast in shoulders are different for left hand and right hand turnouts. Ties in sections having the rail fastened to a steel plate, and then the steel plate fastened to the tie, shall be designed for interchangeable use in left and right hand turnouts.

Ties may be oriented in a fanned layout per Figure XII (b) in which case all of the turnout ties may be used for left hand, right hand, or equilateral turnouts. This minimizes the number of different ties which must be produced, and stocked, but the geometric calculations to accurately locate the fastenings in the fanned layout are considerably more complex.

1.11.2.2 Tie Spacing

Tie spacing in the switch section is usually determined by the location of the various switch mechanisms and connecting rods. Care must be exercised to ensure adequate ballast cribs for tamper tool operation. Satisfactory performance has been achieved where a minimum crib width of 7 inches has been maintained, but recesses may be required in the tie as discussed in Section 1.11.3. Beyond the switch section, 24 inch tie spacing can be used. Some variations may be necessary to ensure correct tie placement at the frog, especially in the case of moveable point frogs.

1.11.2.3 Fastener Placement

To follow the curve of the turnout track, it is common for fasteners to require varying degrees of rotation in the ties. For rotation of the shoulders on ties with elastic fasteners, the method shown in Figure XIII shall be used since it allows the use of standard rail pads, and automatic clip application machines.

Trackwork plates shall be positioned parallel to the centerline of the tie, and the position of all inserts as well as cast in shoulders must be checked to ensure they do not contact the prestress tendons.

1.11.2.4 Ties in Crossovers

Where concrete ties are used in crossovers, either of the methods shown in Figure XIV may be used at the center portion of the crossover to support the closely spaced tracks.

1.11.3 Tie Dimensions

The turnout tie should have a constant cross section over the entire length. In some cases it may be necessary to make recesses or chamfers to fit heaters, machinery, or other attachments. Recesses are often needed in the sides of ties in the switch section, where temperature changes will cause the point rails, connecting rods, and all attached hardware to shift position.

The dimensions in Section 10.1.3 are applicable except as noted in the following sections.

1.11.3.1 Length

The individual tie lengths are calculated based on the turnout geometry. The maximum tie length depends on the track geometry. The minimum tie length must be calculated based on track gage, distance from the rail to the outermost fastening in any tie, and the bond development length of the prestress tendons.

1.11.3.2 Minimum Depth

The minimum design depth of the turnout tie is governed by the design bending moments in Section 1.11.5. Satisfactory performance has been obtained with ties greater than 8.5 inches in depth.

1.11.3.3 Maximum Width

The maximum width of the turnout tie shall not be greater than 12 inches. For tie widths greater than 11 inches, extra care must be exercised to ensure adequate ballast cribs in the switch and frog sections where tie spacing may be reduced, and switch rods, baskets, etc. are located. These elements will also shift position in the cribs as temperature changes.

1.11.3.4 Rail Cant

The turnout ties normally provide for no rail cant. Where a transition must be made to ties with rail cant, it can be achieved through the use of special trackwork plates, special cast ties, elastic fastening systems, or pre-twisted rails.

1.11.4 Design Considerations

Section 1.1.5.2 recommends that continuous welded rail be used on concrete tie track. For turnouts with concrete ties it is also recommended to fully weld the rails to the points and frog, and to locate any joints which may be required over a ballast crib, rather than over a tie.

Field instrumentation may be used to verify loadings in pilot installations, especially in cases where open flangeway frogs are used, or speeds are higher than usual. Instrumentation can also be used to verify support conditions, and the need for maintenance after accumulated tonnage.

1.11.4.1

The minimum pre-compressive stress at any vertical cross section should be 1000 psi after all losses and without any applied load.

1.11.5 Flexural Strength

The minimum unfactored flexural capacity of the turnout ties is shown in the table below, based on the same design considerations as Sections 1.4.1 and 1.4.2. The ties shall be tested in accordance with the Rail Seat Vertical Load Test described in Article 1.9.1.4, with distance 2x/3 taken as 14 inches for this test.

Minimum Unfactored Positive Moment Capacity	390 inch-kips
Minimum Unfactored Negative Moment Capacity	300 inch-kips

1.11.6 Support Conditions

Special care must be given to the support conditions for concrete ties in turnouts to ensure that bending moment capacities are not exceeded. There are many ties where four railseat sections must each have ballast tamped beneath them, and this can cause large negative bending moments when a train passes on any two of the railseats.

Frequently the trackwork in the switch and frog sections covers a substantial portion of the ties, and care must be taken to ensure that ballast is adequately tamped under all ties. Hand tamping may be necessary in some cases to tamp around plated tie sections, heater ducts, or other equipment.

Ties oriented in a fanned layout per Figure XII (b) will require more time for proper surfacing than ties oriented at right angles to the mainline track per Figure XII (a).

At the option of the Engineer, serrations may be considered in the sides or bottom of the tie to increase lateral resistance in the ballast.

Some mechanical equipment may be unable to properly lift and surface the increased weight of concrete ties in turnouts, and assistance with hand jacks may be needed. Also, some mechanical equipment may not be capable of, or may not be set up to tamp ties more than 8 inches deep, and this may cause spalling on the edges of ties. Equipment with sufficient reach must be used to surface concrete turnout ties.

The extra length, and stiffness of concrete turnout ties causes a large increase in vertical track modulus. It is recommended that concrete track ties be used at the three entrance points to concrete tie turnouts, so that the change in track modulus is minimized.

1.11.7 Tolerances

1.11.7.1 Camber

Vertical camber in the ties as laid should not exceed 1/1000 of the tie length. In some manufacturing methods, it may be necessary to check that the horizontal camber does not exceed this value as well.

1.11.7.2 Fastenings

Cast-in inserts and shoulders for fastening systems should be located within ± 1/16 inch of the position shown on the drawing. Angular tolerance should be within 0.5 degrees of the rotation specified.

1.11.7.3 Tie Spacing

Ties should be spaced within ± 1/4 inch of the accumulated distance from the point of switch.

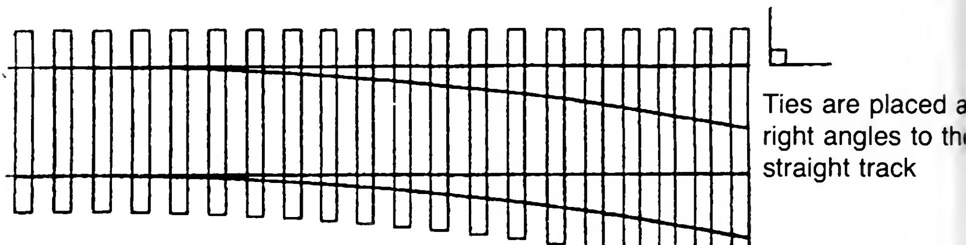


Figure XII(a)

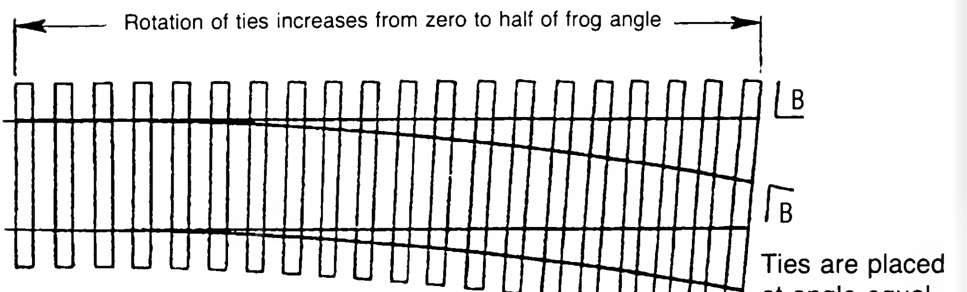
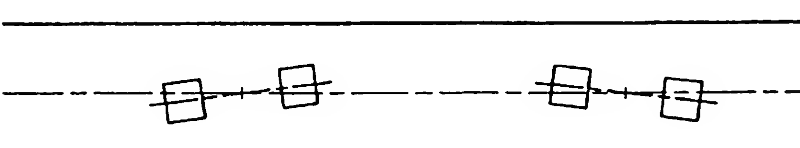
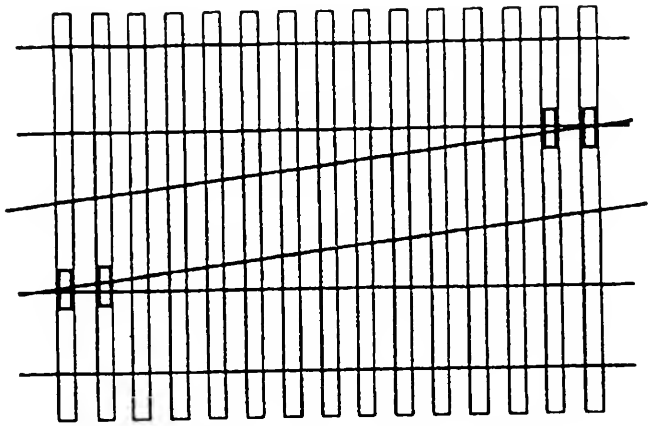


Figure XII(b)

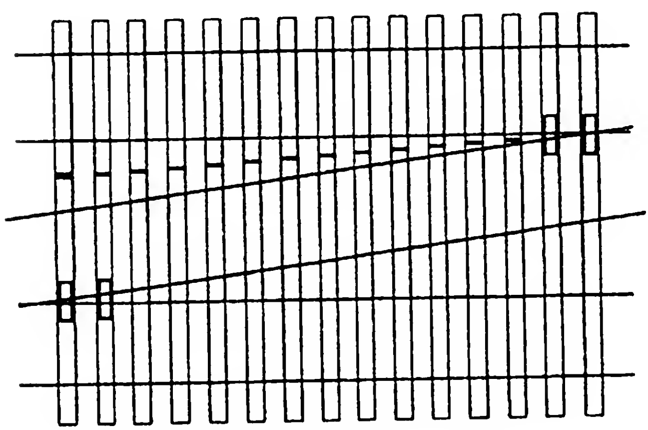


Preferred Rotation Method
for shoulders in Turnout ties

Figure XIII



(a) Long ties span under all tracks



(b) Shorter ties arranged to carry tracks separately

Figure XIV — Ties in Crossovers

APPENDIX I

DUGGAN CONCRETE EXPANSION TEST

The Duggan concrete core expansion test provides a relatively rapid measure of the potential for chemical expansion in concrete. The speed with which expansion occurs in nature depends upon the initial microcrack condition of the concrete, the availability of water, the availability of heat, the degree and frequency of loading stress, and the inherent chemistry of the cement-aggregate combination. Without externally loading the concrete, the Duggan test exaggerates and accelerates the heating/cooling and wetting/drying action of natural environment.

Five concrete cores of nominal 1 in. diameter are wet-drilled from any existing concrete or from proposed job mix concrete which has been cured in proposed fashion for a minimum of 7 days. The cores are end-faced with smooth parallel surfaces to a nominal length of 2 in. The five cores are then placed upright in a plastic container measuring roughly 3-1/2 in. diameter by 4 in. height, and are submerged under 1/2 in. cover with room temperature distilled water. A lid is placed on the container. The core treatment cycle is described below and depicted in Figure 1.

Treatment Cycle

72 hrs. in water at 72°F
Measure core lengths (zero reading)
24 hrs. dry heat at 180°F
24 hrs. in water at 72°F
24 hrs. dry heat at 180°F
24 hrs. in water at 72°F
72 hrs. dry heat at 180°F
Tolerances: Time ± 1 hr.
Temperature ±1.5°F

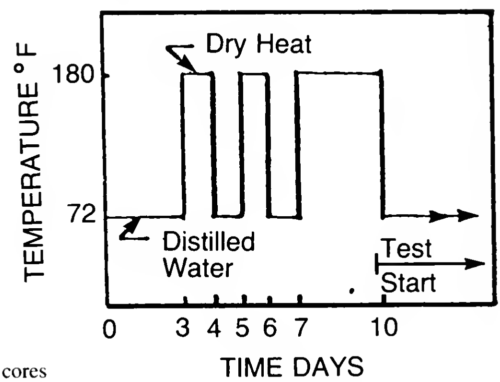


Figure 1. Duggan Core Treatment

NOTE:
After each dry oven heating, bench cool cores for 1 hr. prior to placing back in water. Do not change the water during the treatment or during the subsequent expansion phase of this test.

The first core length measurements (zero readings) are taken at the end of the initial 3-day sc period and prior to any hot oven drying. The reason for this is to include any natural physical expa caused by water uptake at saturation but to exclude any unnatural chemical expansions that may subsequently occur in the cores. Following the final oven heating, the cores are allowed to bench cool for 1 hour and then are placed back in their room temperature distilled water (unchanged). This constitutes the starting point or Day Zero for the expansion test. Core length measurements should then be taken at days 1 and 3, and twice-weekly for the first three weeks. Length changes for the 5 cores should be averaged in terms of percent expansion. Measurements taken on cores drilled from existing bridge abutments ranging in age up to 80 years have indicated that expansion in the Duggan test should not exceed 0.05% at Day 20 for durable crack-free concrete. Until more experience and data is obtained on North American concrete ties, the permissible core expansion at Day 20 should not exceed 0.15%.

Proposed 1993 Manual Revisions To Chapter 14 - Yards and Terminals

In Chapter 14 Sections 2.4, 2.6, 4.2 and 5.2 have been rewritten and are proposed as new material. A summary of significant changes made in the rewrites are:

2.4 Hump Classification Yard Design (Full Automatic Control)

The most significant changes are the expansion of the discussion of criteria and performance and the addition of a presentation on continuous speed control systems, which compliments the presentation on intermittent speed control systems. The material as proposed places emphasis on design considerations and approach rather than theory and mathematical modeling of the railcars movement. However, the stated theory and mathematical formulas are retained for those who may require this information.

2.6 Terminal Design Considerations for Run Through Trains

The proposed material is a new sub-part of Freight Yards and Terminals. It deals with the particular needs for servicing trains that do not need to enter the yard for reclassification. The new material gives a general overview of the run-through operation and train characteristics. The probable servicing functions such as crew change; power change; train change; train inspection; and train servicing are discussed. Design objectives and various design features are presented to guide the work of the facility designer. Graphic examples are provided to assist the reader in their understanding of the material presented.

4.2 Design of Intermodal Terminals

This proposed revision expands the prior materials discussion of intermodal (Trailer-on-Flat-Car and Container-on-Flat-Car) terminals to address the requirements unique to the use of railcars carrying containers in a double stack configuration. In general, the terminology has been updated from "Piggyback," Trailer-on-Flat-Car (TOFC) and Container-on-Flat-Car (COFC) to "Intermodal." Other additions include a discussion of "Roadrailer" terminal needs and equipment, and van carriers for loading and unloading rail cars. Figures 6 through 8 will be added to Figures 1 to 5 currently in the Section.

5.2 Servicing Facilities

Sections 5.2.1 and 5.2.3 have been expanded in this proposed change to better address the many issues surrounding the design of fueling and sanding facilities.

The new rewrites are as follows:

2.4 Hump Classification Yard Design (Full Automatic Control)

2.4.1 General

A hump classification yard should be designed for the volume and character of traffic to be handled and should provide for continuous movement while humping with minimum loss of time between successive humping operations; also for the movement of cars by gravity from the crest to their proper tracks in the classification yard without damaging impacts.

Tracks at the outbound end of the classification yard should be connected to ladders so that classifications normally assembled in one train may be assigned to permit gathering from one ladder, thus providing for minimum movement of trim-end engines. A sufficient number of ladders, with lead connections to departure tracks, should be provided to permit working at least two trim-end engines where required with minimum interference; refer to Plan 1.

Where required, a lead track from the receiving yard to the hump crest can be designed to provide an under-car inspection, set-out track for explosive and hazardous commodities and a connection to release road locomotives. A second track leading from the receiving yard to the hump will permit the use of a second hump locomotive for continuous humping operations. If trains from two or more directions are to be humped in one direction over the hump, provision should be made so that cars can be moved into the end of the receiving yard next to the hump with minimum interference with humping operations.

It may be desirable to make up and dispatch trains from the classification tracks if local conditions permit, and such a method of operation usually expedites movements through the yard and reduces the expense. This requires that a sufficient number of classification tracks be long enough for each to accommodate a full-length outgoing train, or that lead tracks be provided at the outgoing end such that the combined length of a classification track and a lead track be sufficient for a full-length train, thus avoiding unnecessary doubling over or interference with hump operation. This may involve a temporary reassignment of classification during the inspection and preparatory time of a departing train.

Departure tracks may be required for making up and dispatching trains, depending on local conditions.

The average gradient of a track leading to the crest of the hump should be such as to permit pushing the longest and heaviest train at humping speeds consistent with the proposed available power.

A good walkway surface should be provided at the hump crest on both sides of the track for the pin-pullers. If only one pin-puller is to be used then the walkway can be on the right hand side, when moving toward the hump. (It is desirable that cars be uncoupled from the right hand side so that the forward knuckle will be open, as the impact of normal coupling will often close the rear knuckle.)

Adequate lighting will be required throughout the yard.

Access routes to switches, retarders and buildings within the yard may be needed for automobiles, trucks and maintenance vehicles.

Two outer roadways running the length of the yard, and parallel with the tracks can be ideal to facilitate ease of vehicle movements from one end of the yard to the other.

Tracks can be set with extra wide centers between adjacent groups to give access for maintenance vehicles to move into the body of the yard.

The outer and inner roadways can be connected across the yard by constructing level grade road/rail crossings at the narrow ends of the track layout and where the minimum number of tracks need to be negotiated.

For movement across the yard at the hump-end a tunnel may be constructed under the hump itself.

Adequate car parking facilities for employee and company vehicles at the various office and workshop locations should be a consideration.

If the identification numbers of incoming cars are to be read and recorded by a video camera system, then special purpose high density illumination should be provided at the camera location.

Modern automated hump classification yards fall into two principal categories, Intermittent Car Speed control or Continuous Car Speed control systems. A third category can be a hybrid system which combines Intermittent and Continuous control systems.

(i) Intermittent car speed control systems in which powered, electronically supervised clasp retarders are located at discrete positions to control the velocity and progress of the cars traveling through the yard.

The position and speed of the cars is constantly measured, monitored and predicted by the electronic supervisory system which commands the modes of the clasp retarders.

The principal retarders, located in the switching area, are usually powered electrically or pneumatically.

Other types of supplementary retarders may be needed such as tangent point retarders located at the entrances to the classification tracks, and retarders located at the exit ends of the tracks to prevent car run-outs.

The automatic control of the switches, to route the cars into predetermined classification tracks, is invested in the supervisory system along with other operational functions.

(ii) Continuous car speed control systems in which speed sensitive hydraulic retarder units are distributed along all tracks to continuously monitor and control the velocity of the cars traveling through the yard.

This type of retarder is self-contained and needs no exterior power supply or electronic supervisory system.

These retarders are mounted in close proximity along the tracks from the hump, through the switching area and for a selected distance down into the classification tracks.

Arrester zones can be formed by installing this type of retarder at the exit ends of the classification tracks to prevent car run-outs.

An electronic control system is needed for automatic switch operation and to supervise other operational functions.

(iii) A hybrid system that combines an intermittent control system with a continuous control system, 2.4.1. (i) and (ii) above, can be employed to develop a yard having high car speeds in the switching area and accurate coupling speeds in the class tracks.

In such a yard design the velocity and progress of the cars in the switching area would be controlled by an intermittent car speed control system. The function of this part of the hybrid system would be to ensure adequate separation between cars so as to permit movement of the switches for routing; and to predict, monitor and control the speeds for the cars arriving at the classification tracks.

The velocity of the cars in the classification tracks would be controlled by a continuous car speed control system. The function of this part of the hybrid system would be to ensure a maximum allowable coupling velocity of the cars, to promote full car closure in the tracks and to prevent car run-outs from the trim-end.

2.4.2 Intermittent Car Speed Control

2.4.2.1 Hump Control Tower and Buildings

A control building can be located near the hump to house a control tower and offices. This control tower would be positioned to allow the operators a good overall view of traffic movements throughout the yard.

The control tower may need to accommodate a variety of facilities such as:

(a) A control room, at the top of the building, in which to locate a control panel for the manual operation and monitoring of signals, switches and retarders. Operational offices with associated communications, signalling and automatic yard control systems.

(b) Electrical relay room and electrical power supply equipments with their required cable routes and ducting.

(c) Utility services equipment for the building.

(d) Staff amenities accommodation.

A process control system located in the hump control tower can be employed to supervise the greater part of the yard's operational procedures and communications needs. The system would encompass the monitoring and control of the retarders to provide automatic car speed control, and to sequence the movement of powered switches for automatic route selection to direct the cars into designated classification tracks.

2.4.2.2 On Track and Trackside Equipment - Refer to Diagram I

To support a central process controller it may be necessary to install a variety of peripheral hardware at locations on the track.

(i) Car identification equipment. This may be a video camera system or an electronic transmitter/receiver unit to interrogate car mounted identify transponders.

(ii) Hump signal. To control train movements toward the hump crest.

(iii) Car monitoring equipment. Pole mounted photo electric cells and track mounted wheel detectors to monitor car cut lengths. A weigh rail installed in the track to measure axle loads. Electrical track circuits to form a rollability test section. Radar speed detectors to monitor car speed.

(iv) Track scale. May be needed for the commercial weighing of cars. Weight information can also be for input to car speed control system. The scale would need to be installed on a suitable concrete foundation.

(v) Car speed control. Switching area and tangent point retarders mounted in the track to control the speed of cars at strategic locations. These can be electrically or pneumatically powered.

An air compressor house or an electrical supply facility to power the type of retarders chosen can be constructed in the vicinity of the retarders.

(vi) Switches. Powered switches would be needed to route the cars from the crest into the classification tracks. Electrical track circuits or proximity loops, and/or wheel detectors can be included in the switching area to monitor the progress of the cars and provide switch movement protection as required.

(vii) Distance to couple. The classification tracks may be equipped with electronic circuits to determine the distance a car must travel to couple.

(viii) Cable routes. All the above signalling and monitoring equipment would require electrical cabling enclosed in trenches, troughing, conduits or directly buried.

Track side electrical equipment cases would be needed at various locations.

(ix) End of track retarders. These may be used at the trim-end of the classification tracks to prevent car run-out.

2.4.2.3 Trim-End Tower

In large yard developments with extended classification tracks a trim-end tower building may be required to house an elevated signal and traffic control room from which operations in the departure end of the yard may be supervised.

The trim-end tower may need to contain a number of facilities such as:

(a) A signal and control room with allied signalling and communications equipment.

(b) Electrical relay room and electrical power supply equipment with their required cable routes.

(c) Utility services equipment for the building.

(d) Staff amenities accommodation.

2.4.3. Continuous Car Speed Control

2.4.3.1. Hump Control Tower and Buildings

A control building can be located near the hump to house a control tower and offices.

The functions of this building could be similar to that described in 2.4.2.1. with the following exceptions:

(a) There would be no console or equipment for the manual control of the retarders.

(b) There would be no electrical equipment or process control system for the automatic control of the retarders.

2.4.3.2. On Track and Trackside Equipment - Refer to Diagram II

(i) Car identification equipment as per 2.4.2.2.(i).

(ii) Hump signal. To control train movements toward the hump crest.

(iii) Continuous car speed control system. Hydraulic type retarders bolted to rails at close intervals throughout all the tracks.

(iv) Switches as per 2.4.2.2.(vi).

(v) Cable routes. The signalling and track circuit equipment would require cabling as per 2.4.2.2.(vii).

2.4.3.3. Trim-End Tower

In large yard developments a trim-end tower may be required as described in 2.4.2.3.

2.4.4. Hybrid Car Speed Control System.

2.4.4.1. Hump Control Tower and Buildings.

A control building can be located near the hump to house a control tower and offices.

The description and functions of this building would be similar to that described in 2.4.2.1.

2.4.4.2. On Track and Trackside Equipment

To support the Intermittent Car Speed Control part of the system it may be necessary to install in the switching area a variety of peripheral hardware at locations on the track.

For a description of the type of equipment that may be included refer to 2.4.2.2. (Note: Item (vii) and item (ix), distance to couple circuits and end of track retarders respectively would not be required.)

For the Continuous Car Speed Control part of the system hydraulic type retarders would be needed in the classification tracks as described in 2.4.3.2.item (iii).

2.4.4.3. Trim-End Tower.

In large yard developments a trim-end tower may be required as described in 2.4.2.3.

2.4.5 Objective

The objective for constructing and equipping an automated hump yard is to facilitate an efficient and expedient method of automatically routing free running cars into designated classification tracks for the formation of outbound trains.

To achieve this objective it is necessary to meet certain design criteria within the overall concept.

2.4.5.1 Design Criteria

(a) To provide a hump of sufficient elevation to ensure that all cars, having a practical rollability value will penetrate far enough into the classification tracks to achieve a high percentile of closed couplings. It may be necessary to relax this requirement under severe weather conditions such as extreme cold, snow or high winds; but the minimum need is for all cars to run beyond the clearance points.

(b) To form accelerating gradients from the hump that will promote separation between successive cars to facilitate the operation of switches between cars.

(c) To form a series of gradients throughout the switching area of the yard so that the car speeds are compatible with the specified humping rate (car throughput) and with the chosen retarder system.

(d) To automatically control the velocity and destination of the cars by providing car retarder and route selection systems respectively.

(e) To form gradients in the classification tracks that will assist the cars to penetrate the tracks fully and couple at 4.0 mph maximum.

2.4.5.2 Design Methods

Although it is the car rollability band width that influences the gradient profile of a yard, it is the retarder system that assumes the prime role in yard design by the fact of measuring and monitoring the car speeds to achieve the desired throughput, controlling acceleration, maintaining separation in the switching area and determining car performance in the classification tracks.

Because of this important role in the suppliers of retarder systems have, over the years, acquired comprehensive computer aided programs with which to design the profiles and plot car performance curves for a yard based upon fundamental equations of energy and motion, accepted constants that affect car behavior and specified variable parameters.

2.4.5.3 Typical Retarder Control Systems

2.4.5.3.1 Intermittent Control System

(a) In an automatic yard employing intermittent retarders and a process controller system the cars are weighed and classified after leaving the crest of the hump. Rollability measurements are taken on test sections of track on the approach to the primary retarder (tangent track rollability) and on the approaches to the group retarders (curved track rollability). This information is stored for reference in predicting the car exit velocities from the group retarders.

The computed value for the tangent rollability is interpolated with that for the curved rollability to provide a modified value which will be used to predict the car exit velocities from the group retarders.

(b) The primary retarder is used to adjust the velocity of the cars in order to maintain adequate separation between them; and to assist the speed control function of the group retarders by providing suitable exit velocities from the primary retarder.

As a car passes through the primary retarder, the braking shoes are applied at the maximum pressure allowable for the car's weight category. Radar units measure the speed of the car moving through the retarder and transmit information to the process controller in the form of a servo loop to continuously monitor car speed and retardation force.

(c) The speed control method is the same through the group retarders as for the primary retarder except that in this case the exit velocities must cater for the cars running varying distances down the classification tracks to finally couple at 4 mph maximum.

The rollability value of the car, based upon the information collected at the tangent and curved rollability test sections, is modified in accordance with track, car weight and car type characteristics. The track resistance characteristics are determined from computer models and practical tests made prior to the system being operational.

The classification tracks are equipped with electronic distance to couple circuits which monitor the positions and speeds of the cars and transfer this information to the process controller, from this the exit velocity from the group retarder is determined for each car. This exit velocity will be automatically and continually modified during switching operations to strive to achieve the maximum performance in closed couplers and 4 mph maximum speed.

(d) For some yards, where the distance from the group retarder to the tangent points, and the distances to couple are extensive, it is necessary to employ tangent point retarders to attain the required car performance in the tracks.

The exit speed from the group retarder is then controlled so that the cars arrive with predicted velocities at the tangent point retarders. These retarders, being radar equipped will monitor and control the car speed in accordance with the distance to couple information.

(e) The clasp type of retarders used in intermittent car speed control systems act upon the sides of wheels. The brake shoes apply a frictional force to slow the wheels of the car; this force is controlled in increments that are proportional to the car weight classes.

Variations of these types of retarders are:
Electrically actuated, spring powered
Pneumatically powered
Hydraulically powered
Hydraulically actuated OFF, and mechanically powered, via a lever system, by the weight of the car wheel - the retardation force is weight proportional.

A primary or group retarder can be of various lengths and is installed on a well constructed and consolidated foundation. The mechanical components and associated steelwork are integrated in assembly with special ties that have custom made supports on which the running rails are attached within the retarder. This type of retarder, due to the frictional action at the brake shoe to wheel interface can in some instances emit loud noise levels of high frequency. Dependent upon location, it may be an environmental requirement to construct acoustical barriers in close proximity to the retarders.

(f) Typical gradients associated with this type of yard are illustrated in Fig. A. In the design of the track profile for a classification yard, the gradients will depend upon factors such as car throughput, car rollability bandwidth, track curvature and turnouts, and local weather conditions.

2.4.5.3.2 Continuous Control System

(a) In a yard employing the continuous control method the car velocity for the switching area, i.e. from the hump to the tangent points, is selected during the yard design stage. This switching area velocity is dependent upon the humping rate, separation between cars, car rollability bandwith, range of wheel diameters, the track characteristics and the length of the switching area. The hydraulic retarder units are then calibrated during manufacture to control all cars constantly at this selected velocity.

(b) The hump, for this type of yard will compromise concurrent concave and convex vertical curves and finish at the first switch.

The hump is used to accelerate cars to the switching area velocity and the installation of the retarder units commences in the sag curve at the point where the minimum rollability car attains the switching area velocity.

(c) A constant gradient is formed from the first switch to the tangent points in the classification tracks. This gradient is designed for a modified rollability value comprising car rollability plus air, wind and track characteristic resistances. These characteristics, together with the maximum car weight will determine the quantity of retarder units needed to provide continuous speed control.

(d) At the tangent points, or in some instances the clearance points, deceleration zones are used to slow the cars from the switching area velocity to a 4.0 mph coupling velocity and are situated on the initial gradients at the beginning of the classification track.

The quantity of retarders needed for each zone will depend upon the change of velocity required, the maximum car weight and the initial classification track gradient.

(e) Typically, the initial classification track gradient can continue for approximately one third of the total track length with retarders installed along the track to prevent the heavy, low rollability cars from accelerating above 4 mph. This initial gradient will assist the penetration of cars down into the tracks and provide a high percentile of coupling.

(f) The hydraulic retarders used in continuous control systems are relatively small units installed at close intervals along the tracks. They are fixed to the inside of the running rails and actuated by the wheel flange.

(g) Typical gradients associated with this type of yard are illustrated in Fig. B. In the design of the track profile for a classification yard the gradients will depend upon factors such as car throughput, car rollability bandwidth, track curvature and turnouts, and local weather conditions.

2.4.5.3.3 Hybrid Control System

The formulation of a hybrid system of car speed control is based upon the use of the clasp type of retarders, with process controller, in the switching area to perform the duties of maintaining separation and controlling the group retarder exit speeds; and a continuous control system that commences with deceleration zones in the classification tracks and continues with coupling speed control zones.

The intermittent control system in the switching area would be as described in 2.4.5.3.1 (a), (b) and (c) with a modification to the group retarder exit speed requirements, and the distance to couple circuits would not be needed. The group retarder exit velocities would be controlled to provide a bandwidth of velocity for the cars arriving at the deceleration zones, with the lower limit of velocity being applied to the heavy cars and the higher limit to the light cars in order to produce a zone that is economic in retarders; and also to ensure good penetration of the light cars through the zone.

2.4.5.4 Design Parameters

In preparing for a classification yard design it is necessary to ascertain the parameters.

(a) Car throughout, the rate at which cars will be expected to pass over the hump. This can be expressed as the humping velocity by considering all cars to be of average length.

(b) The vertical convex and concave curves for the hump profile should be specified in order to ensure adequate clearance from the car structures and prevent binding of car knuckles.

(c) The maximum and minimum car weights should be stipulated in association with car types, length and wheel diameters.

(d) Details of the weigh scale length should be ascertained, with the minimum response and record times, in order to be able to specify the minimum time that a car must occupy the scale to produce valid recordings.

(e) One of the most important parameters is the rollability bandwidth for the variety of cars to be humped. Detailed research and analysis should be undertaken to determine practical values. A good source of information is the printouts from existing control systems that are already operating in established yards. The basic tangent rollability values for the total car population should be ascertained and specified.

(f) In the event of a catch-up, i.e. zero separation distance between leading and following cars, the movement of the automatic switches is locked in position to prevent derailments. Various types of electrical switch protection circuits can be employed to guard the switches. In order to be able to specify the minimum separation distance between cars it is necessary to ascertain details of the circuits, such as occupation length and response time, along with the response and operating times for the switch machines.

(g) It is the usual practice, where clasp retarders are used, to provide sufficient retardation so as to be able to stop the heaviest car in the event of an emergency in the yard, a derailment for instance. This requirement can be catered to by designing for both primary or group retarders alone to be capable of stopping a car. A more economic solution, and the one usually adopted, is to design for the primary and group retarders in unison to have sufficient retardation to stop the heaviest car. The preferred requirement should be specified.

In yards employing a continuous speed control system it is not possible to cater for emergency stopping; but because the car speeds are relatively slow, avoidance action can be taken by manually routing cars away from danger.

(h) The geometric data of curves and turnouts for all tracks will need to be specified for a well designed yard layout. In designing the layout the curves and turnouts should be as generous as possible in order to reduce the total car rolling resistance to a minimum.

Additional gradient, to compensate for curve resistance, may be added with advantage to the long curves that lead to the outer groups of tracks.

Standard turnouts should be preferred to any of special design as these may not be readily available in a future emergency if a replacement switch panel is needed.

(j) In the interest of safety a combination of end track retarders or arresters, with a plus track gradient, should be considered to avoid car run-outs. The retarder, or arrester capacity must be designed to stop a heavy car at a specified maximum velocity. In theory, the maximum velocity should be 4 mph, the coupling velocity; but in practice, with intermittent control systems, cars will at times arrive at the trim-end travelling in excess of 4 mph.

2.4.5.5 Theory

(a) Car velocity

The velocity of a car travelling along a gradient can be determined at any point by the equation: $V^2 = 2gh$, where:

V = car velocity, ft/sec.
g = gravitational acceleration, i.e. 32.2 ft/sec^2.
h = energy head, ft.

The energy head, h can be the potential energy, due to the elevation on a gradient, that will accelerate a car to velocity, V (ignoring resistance losses); or it can be the velocity head, in which case it is the energy invested in the car velocity; for clarity, let H ft. = velocity head and h ft. = potential head, refer to example 2.4.5.5.(d).

This basic energy equation needs to be modified to include two coefficients that affect the movement of a car, these are:

The coefficient of rollability, R.
The coefficient k, to allow for the rotational kinetic energy of the wheel sets.

(b) Rollability

The rollability (rolling resistance) of a car can be expressed as a coefficient, a force per weight ratio or an equivalent percentage gradient, i.e. 0.003 = 6.0 lb/ton = 0.3%. This expression states that a car with a rollability coefficient of 0.003, or 6.0 lb/ton resistive force, would travel with constant velocity on a 0.3% gradient tangent track. The total rollability value for a car is the sum of the tangent rollability + curve and turnout resistance + air and wind resistance. Typical rollability coefficients are:

Tangent rollability = 0.0005 min. to 0.006 max.
Curve resistance = 0.0004 to 0.0005/degree
Air resistance = 0.00016 per ft./sec. velocity
Wind resistance = 0.0001 per ft./sec. wind velocity

Note: In a Continuous Speed Control system an additional factor must be introduced to allow for the idling resistance of the retarder units when operating below their threshold control speed.

(c) Rotational kinetic energy

The coefficient $k = 1 / [(1 + (4wr^2) / (D^2W)]$, where:

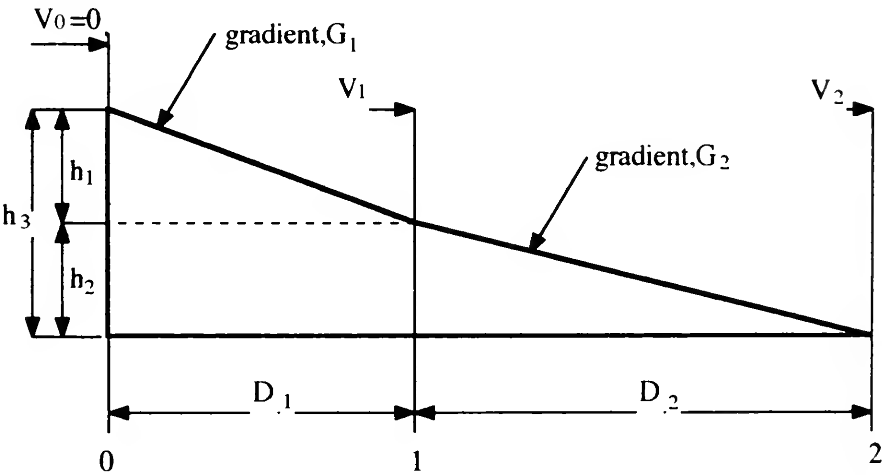
w = weight of wheel set, lbs.
r = radius of gyration of wheel set, ins.
D = wheel tread diameter, ins.
W = Car weight, lbs.

For an estimate of car performance on a given gradient a simplified value for k can be determined from:

$k = (W + 2) / W$, for a 4 axle car.

Typical car weights are 270,000 lbs. max. to 40,000 lbs. min.

(d) To determine change in velocity:



Let: V_0, V_1 & V_2 = velocity, ft/sec.
H_0, H_1 & H_2 = velocity head, ft.
h_1, h_2 & h_3 = potential head, ft.
D_1 & D_2 = distance, ft.
G_1 & G_2 = gradient coefficient
R = total rollability coefficient

To determine V_1: $V^2{}_1 = (2\ g\ H_1) / k$

$H_1 = H_0 + h_1 - (D_1\ R)$

also $H_0 = 0$ and $h_1 = (D_1 G_1)$

subs. $H_1 = D_1\ (G_1 - R)$

then $V_1 = \sqrt{[(2\ g\ H_1) / k]}$ ft. / sec.

To determine $V^2{}_2$: $V^2{}_2 = (2\ g\ H_2) / k$

$H_2 = H_1 + h_2 - (D_2\ R)$

also $H_1 = V^2{}_1\ k / 2\ g$ and $h_2 = (D_2\ G_2)$

subs. $H_2 = H_1 + D_2\ (G_2 - R)$

then $V_2 = \sqrt{[(2\ g\ H_2) / k]}$ ft. / sec.

(e) Car separation

The lengths of the Intermittent Control retarders and the weigh scale, and the safe operation of the switches make it necessary to predetermine the separation of the cars as they travel from the crest of the hump to clearance points in the class tracks.

Note, with a Continuous Control system only the separation needed to operate the switches has to be considered and as all cars travel, after the initial acceleration, with approximate constant velocity, on a constant gradient, the design for separation is relatively simple.

(f) Time/Distance curves

Each car must be accelerated away from the hump to produce adequate separation distance between cars and this distance must be maintained at a minimum length throughout the switching area. In order to study and analyze the car's performance and separation it is necessary to compute Time/Distance curves and to introduce retardation at critical points in order to adjust speeds and maintain separation.

In order to design for a "worse case" situation it is necessary to take into consideration the separation changes between a light, high rollability car when followed by a heavy, low rollability one, each routed to adjacent class tracks. There must be adequate separation down to the last level of switches; and finally, a following car must not coincide with a leading one until after the clearance points.

For Intermittent Control systems, retarders must be located at the critical points of the Time/Distance curves in order to adjust car speeds and prevent catch-up between cars of varying rollability values. The exit speeds from the group retarders must be varied to match the distance the cars must run to couple in each class track; when a track is nearly full these exit speeds will be relatively slow and this must be allowed for in the Time/Distance curve by plotting a heavy, low rollability car followed by a light, high rollability car that may need to run unretarded; refer to the sketch of Time/Distance curves in Diag. III.

In Continuous Control systems the car velocity, after initial acceleration, will be nearly constant with little change in separation; it is however, necessary to allow for a speed control bandwidth due the variation in wheel diameters; refer to the sketch of Time/Curves in Diag. IV.

(g) Retardation

A typical Intermittent retarder yard will comprise a master retarder and a number of group retarders: the master will be situated near the hump and its function is to adjust speeds for separation control. The groups, located at the end of the switching area gradient, control the speeds of cars entering the class tracks; their prime function is to release cars at predicted speeds in order to achieve 4.0 mph coupling at varying distances down the tracks. This method of operation is often referred to as "target shooting" and employs Distance to Couple circuits in the tracks, combined with computed exit velocities from the groups. If tangent retarders are used at the entrance to the tracks, then the groups will "target shoot" to these and the tangent retarders will then control the final distances and coupling speeds.

The minimum retarder energy necessary between the hump crest and the clearance points for car speed control is:

Retarder energy = $(H_2 - H_1)$ (Max. weight car), ft. ton; where:

H_1 = Velocity head of low rollability car at clearance
H_2 = Velocity head of high rollability car at clearance

In a Continuous Speed Control system the retarder units are installed at regular intervals throughout the switching area and for distances down into the class tracks. The quantities of retarders needed to provide speed control are dependent upon the control velocity, and are directly proportional to the effective gradient (gradient minus total rollability) and the maximum car weight.

Retarder density = $A(G - R_{min}) / E$, units / ft. where:

A = Maximum axle load, ton
G = Gradient coefficient
R_{min} = Minimum total rollability coefficient.
E = Retarder energy, ft.ton / unit at specified control velocity.

At the tangent points the retarders are installed in dense banks, forming deceleration zones to slow the cars from the switching area velocity down to a 4.0 mph coupling speed.

Quantity of retarders/zone = $A(V^2_{SA} - V^2_{CV})k / (2gE)$, where:

V_{SA} = Switching area velocity, ft./sec.
V_{CV} = Allowable coupling velocity, ft./sec.

As a slight accelerating gradient is usually extended down into the class tracks it is necessary, in order to maintain a coupling speed of 4.0 mph maximum, to continue with a speed control section comprising an appropriate quantity of retarder units. The retarder density can be determined by applying the formula used above to calculate unit density in the switching area.

2.4.5.6 Bibliography

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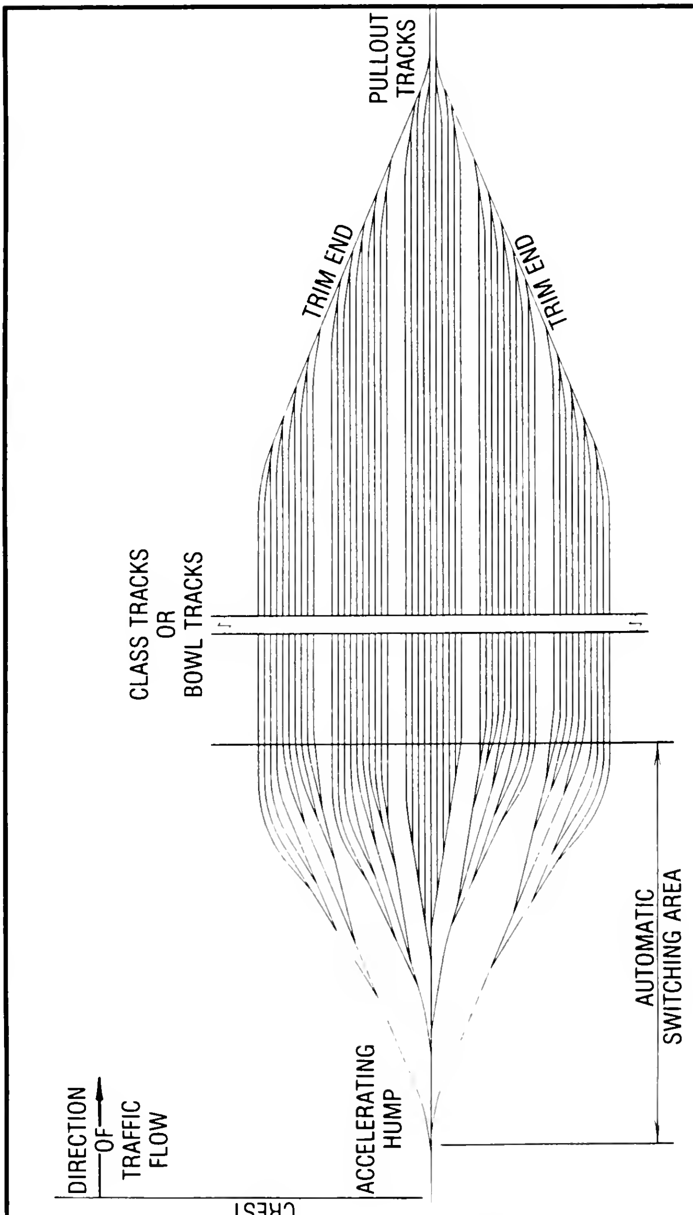
ii. Assessment of Classification Yard Speed Control Systems, SRI.

iii. Proceedings of the Workshop for Classification Yard Technology, May 1980, report FRA/ORD-80/17

iv. Proceedings of the Second Workshop for Classification Yard Technology, May 1981, report FRA/ORD-81/41.

v. Railroad Classification Yard Technology Manual, FRA/ORD-81/20 I, II & III.



Plan 1. Typical Classification Track Layout

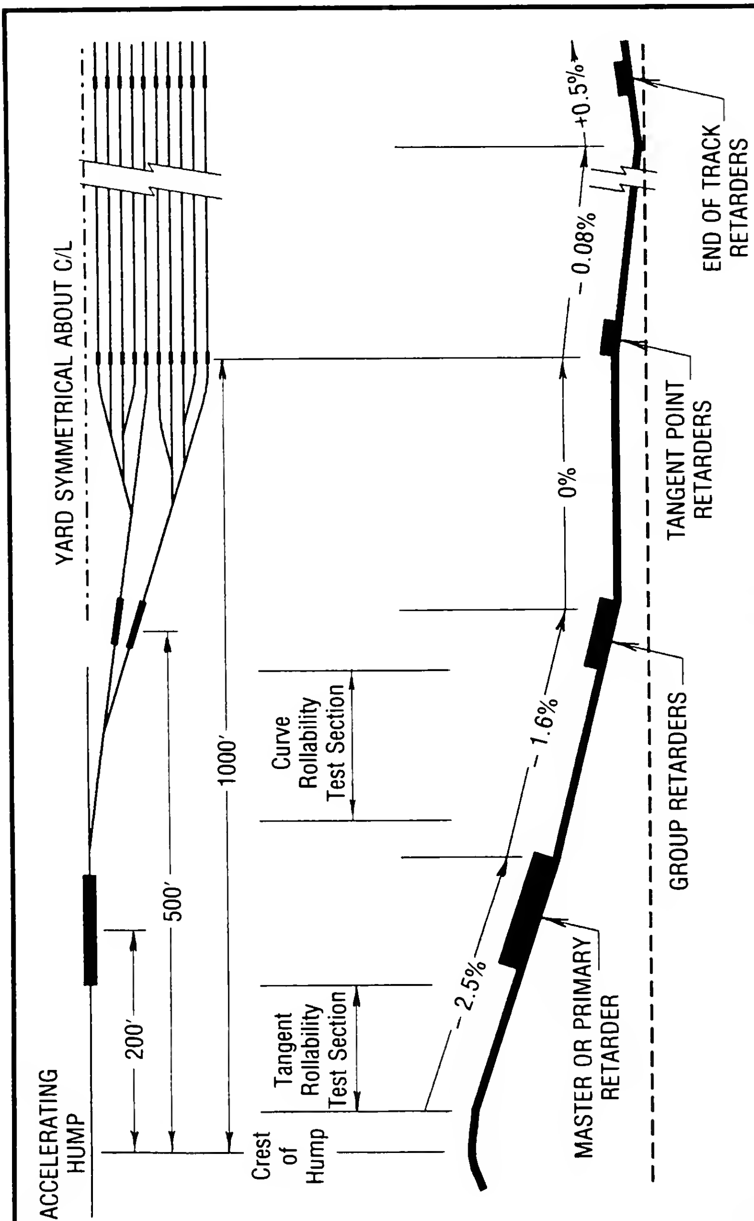


Figure A. Track and Profile Diagram (Intermittent Control)

NOTE: GRADIENTS AND DISTANCES ARE TYPICAL ONLY

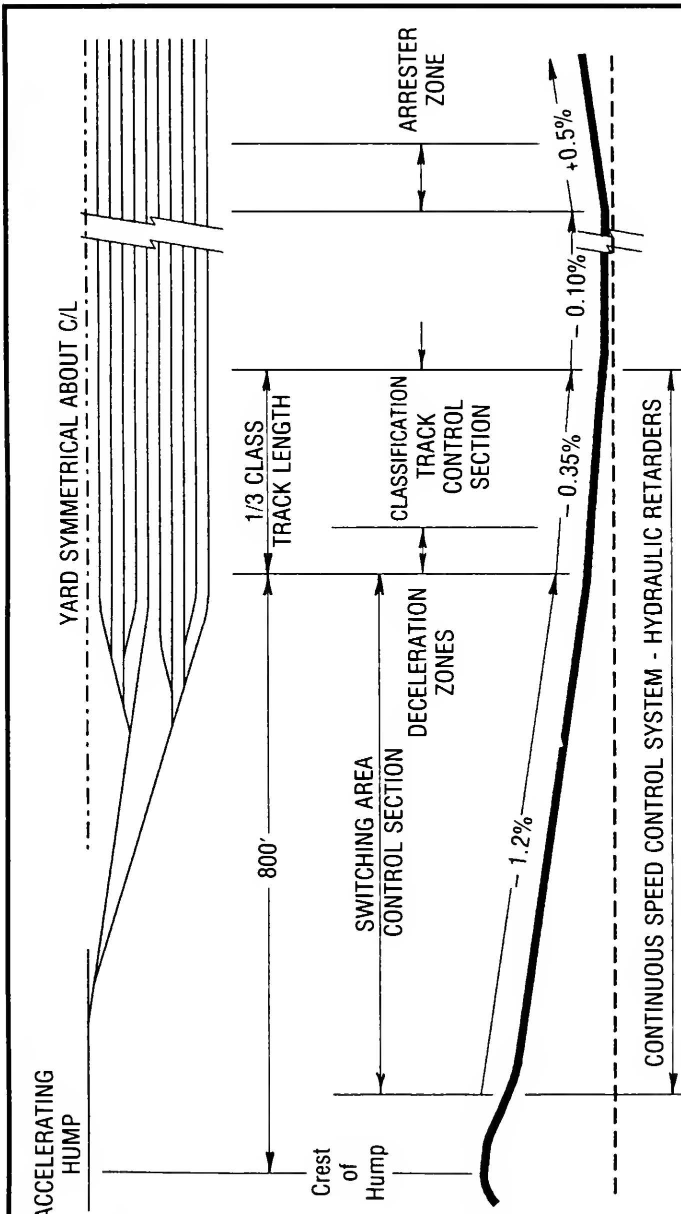


Figure B. Track and Profile Diagram (Continuous Control)

NOTE: GRADIENTS AND DISTANCES ARE TYPICAL ONLY

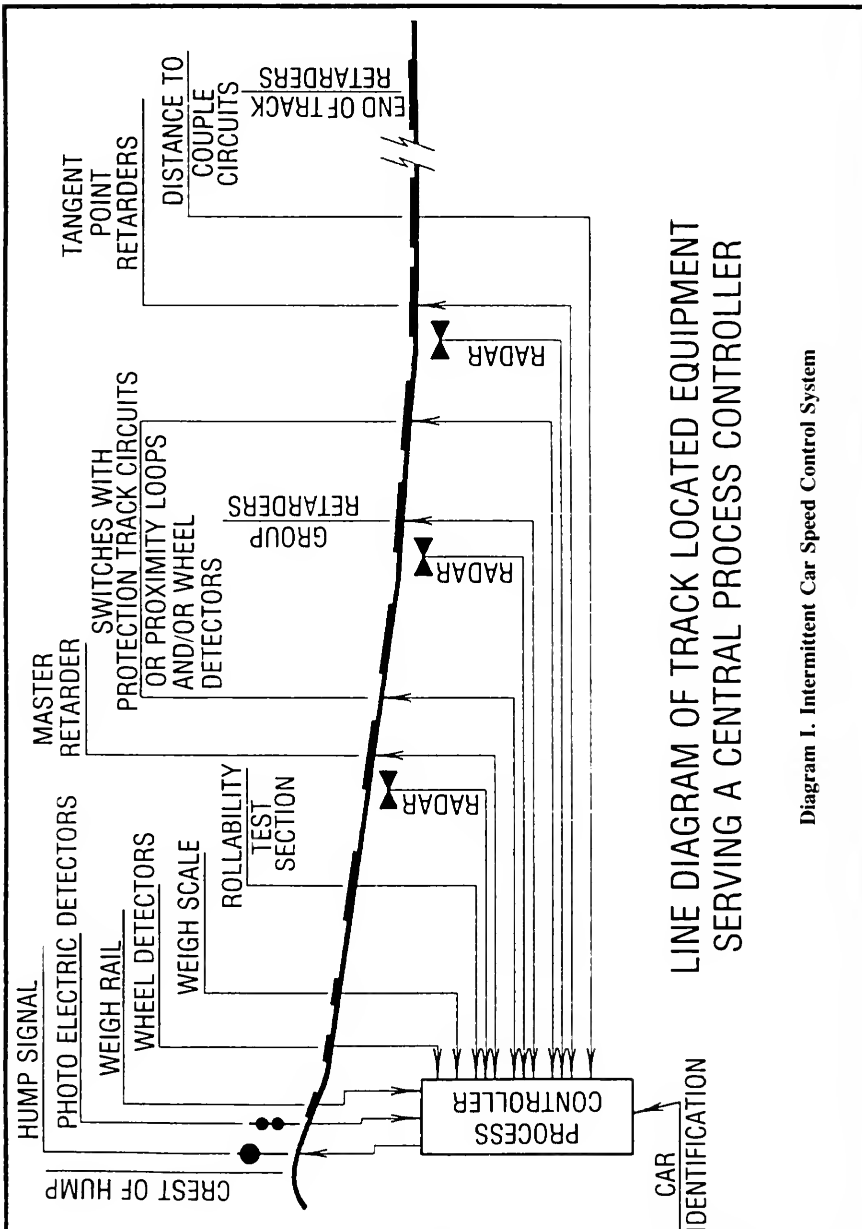


Diagram I. Intermittent Car Speed Control System

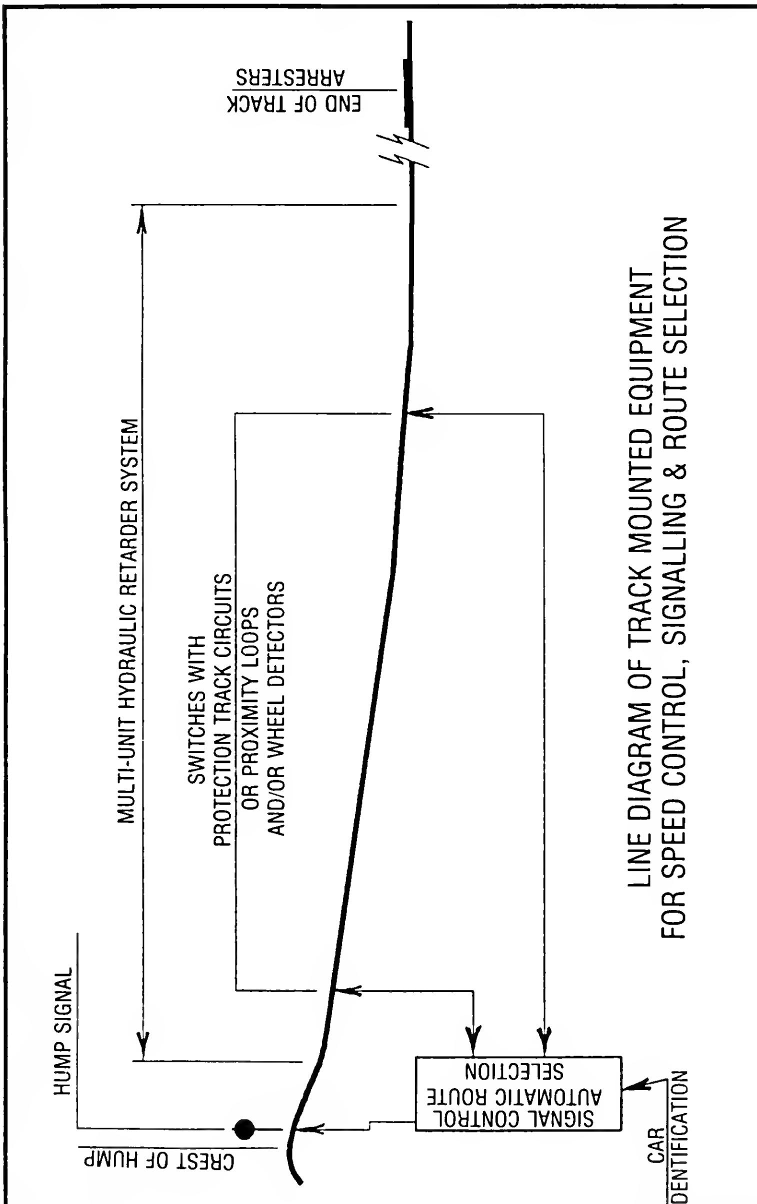


Diagram II. Continuous Car Speed Control System

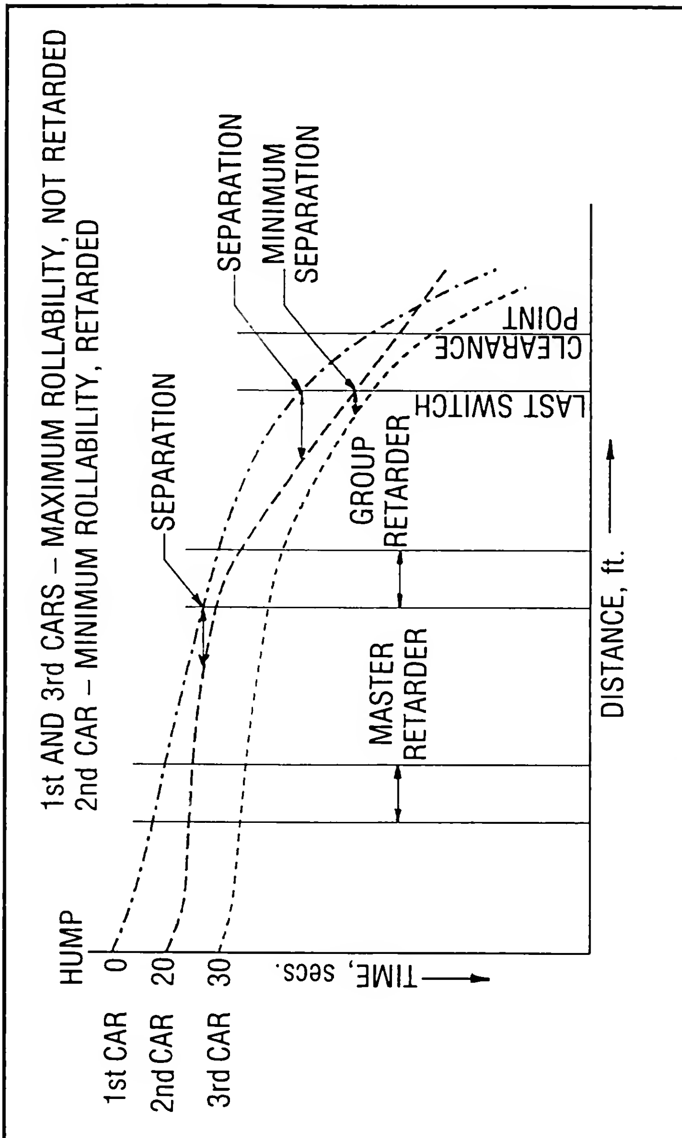


Diagram III. Sketch of Time vs. Distance Curves (Intermittent Control)

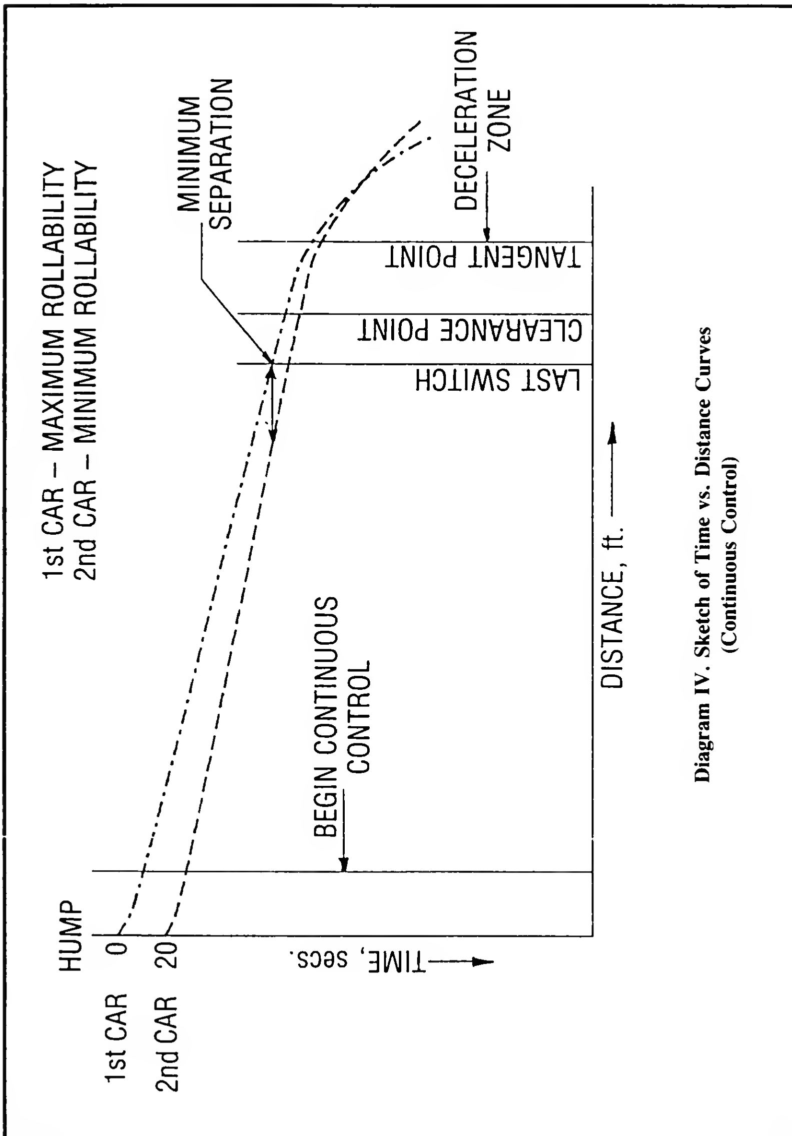


Diagram IV. Sketch of Time vs. Distance Curves (Continuous Control)

2.6 TERMINAL DESIGN CONSIDERATIONS FOR RUN THROUGH TRAINS

2.6.1 Characteristics of Run Through Trains

Run through train operations involve the handling of service from the train origin to train destination with by-pass of normal intermediate yard humping or reclassification. Many variations of the definition for a run through train exist in current rail operations. "Pure" run through trains operate from the shipping origin as a protected "unit" to the receiving destination on a loaded cycle. Many times the train is assigned as a dedicated train set and cycles from origin to destinations. Variations in train operations for unit run through trains include:

Single Origin – Various Local Destinations
Various Local Origins – Single Destination
Various Local Origins – Line Haul – Various Local Destinations

2.6.2 Run Through Train Operation

The operation of these trains in a terminal will likely have an impact on the support yards' efficiency, depending on the required handling of the particular run through trains. In addition, the handling of these trains likely will impact main track operations in and around the terminal. It is important, therefore, that as terminals are laid out or reconfigured that consideration be given to minimizing the impact on yard and main track operations.

2.6.3 Yard Facility Functions

The primary functions involved in the handling of the by-pass or run through trains at a yard facility are crew change, power change, train changes, train inspection and train servicing.

2.6.3.1 Crew Change

Terminals are likely locations where run through trains change crews. The timing of these changes will be affected by the amount of work to be done with the run through train in the terminal.

2.6.3.2 Power Change

Generally, run through train power consists are handled without change at intermediate terminals. Exceptions to this would be power requiring change due to failure on line of road or power change necessary to handle route alignment or train tonnage alteration. Power requirements for the departure trip route is governed by increase or decrease in ruling grades.

2.6.3.3 Train Changes

Generally run through trains are handled as a unit from origin to destination. Exceptions to this would be short unit trains such as grain trains that will be filled with additional blocks at intermediate terminals. Other conditions affecting train changes would be ruling grade restrictions requiring reduction and filling of the train on either side of the grade. In these cases, set-out and pick-up tracks should be made available adjacent to tracks occupied by the run through train.

2.6.3.4 Train Inspection

Inspection of run through trains may be necessary at the terminal depending on where the train originated or other FRA and local requirements. This is accomplished through a vehicle or walking inspection as necessary.

2.6.3.5 Train Servicing

Run through train power may be serviced intact. Those that are may have fuel, sand and water added as well as supplies for the locomotive. The end of train device (EOT) may also be serviced.

2.6.4 Design Objectives

2.6.4.1 Access to Main Lines

Where run through trains are routed or held on other than main tracks, those by-pass tracks should be adjacent to the main line. Turn-outs should be designed to provide entrance and exit to these tracks at 25 to 30 MPH to minimize train delay. Power or spring switches should be considered at these locations.

2.6.4.2 Access to Main Yard

Consideration should be given to the proximity of where the run through trains are routed or held to the main yard. Cars for pick-up or set-out for the run through trains will likely travel through the main yard or nearby support yards. Movement to and from these yards should cause minimal impact to the main line operation.

2.6.4.3 Access to Crew Office

Crew change locations for run through trains should be close to crew facilities to minimize delays. Consideration should be given to roadway access at these locations to minimize trains blocking crew vehicles and provide easy turnaround.

2.6.4.4 Access to Locomotive Shop

Track layout should provide direct access to the locomotive shop, where applicable, for power change-out on run through trains. Route should minimize delays due to yard or main track movements.

2.6.4.5 Access to Car Shop

Consideration should be given to the proximity of the Car Maintenance facility. Bad order cars on run through trains will require placement and pick-up for the nearest car shop. Train inspection personnel may likely be headquartered at the maintenance facility as well.

2.6.4.6 Train Inspection

Inspection roads and access should be provided to allow for both rolling and walking inspection of the through train. Inspection access should provide the ability to perform light repairs to the intact train if the repair condition can be handled without switching.

2.6.4.7 Train Servicing

Train servicing facilities generally include access for power consist fueling and spot maintenance. The run through handling of unit trains may require power or car set-offs. Consideration should be given to allow for switching tracks at both ends of the holding tracks assigned for run through trains. Also servicing of end of train devices (EOT) should be considered.

2.6.5 Design Features

2.6.5.1 By-pass Yard and Siding Tracks

These tracks should be designed to handle the maximum train length. They should be accessed through standard lead ladders with turnouts sized to permit 25 to 30 MPH speeds. The rail in these tracks should be sized to permit these track speeds as well. Where expected train volume would warrant power or spring switches they should be considered.

2.6.5.2 Engine Tracks

Consideration should be given to providing trackage for temporary locomotive storage. This trackage could be utilized to stage locomotive changeouts or for fueling and servicing locomotives. It should be in close proximity to the by-pass yard.

2.6.5.3 Fueling and Servicing

The requirements should be considered for run through train power. A fueling station on the engine track may be necessary to provide quick access to fuel and light engine service, including locomotive supplies. It may be feasible to fuel and service at the locomotive shop or by a mobile truck. For any of these options, ease of access, proper fueling equipment, environmental protection and protection of employees working on engines should be considered.

2.6.5.4 Yard Air

Yard air may be required on the by-pass tracks for expediting train movement. A review should be made of the type of car set-outs and pick-ups and the duration these train blocks will be required to await movement.

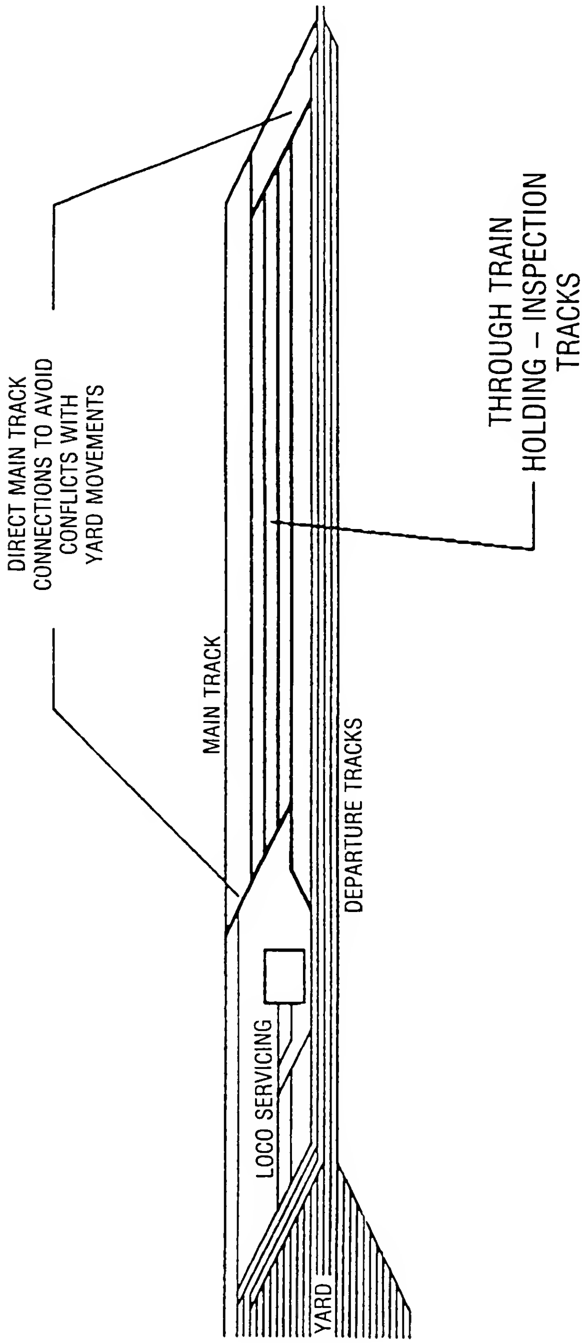


Figure 1. Example Layout

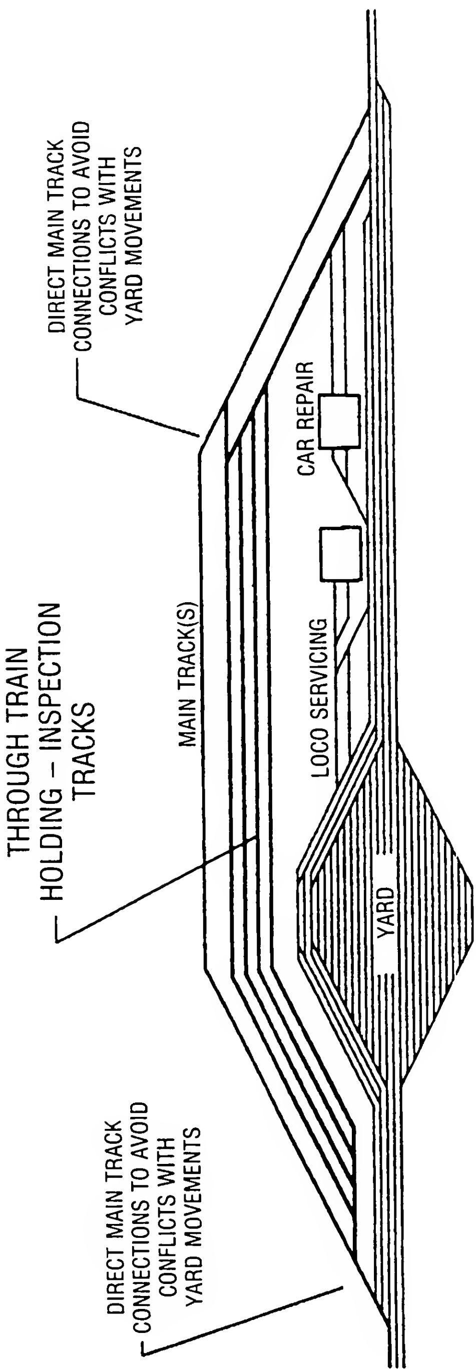


Figure 2. Example Layout

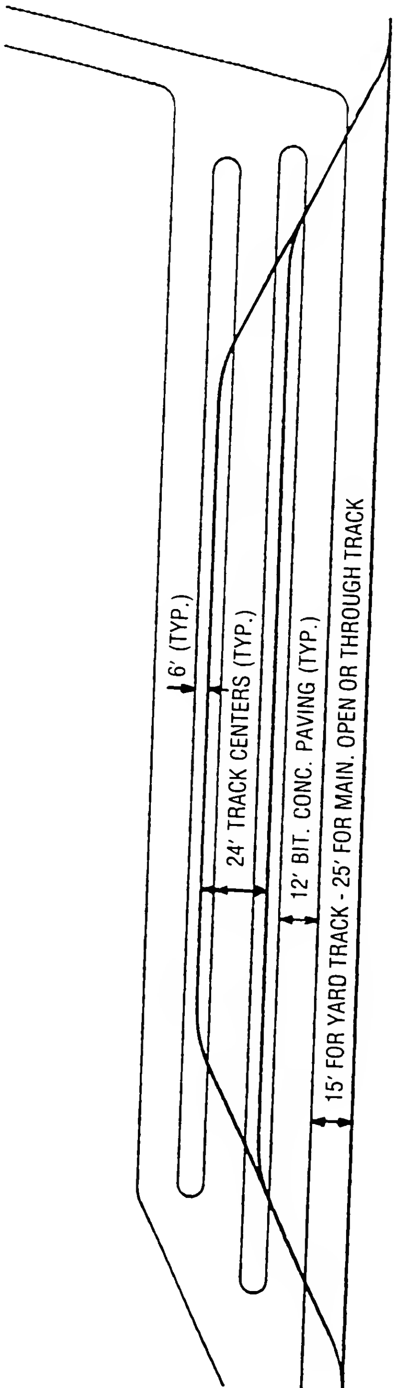


Figure 3. Through Train Track Layout

2.6.5.5 Roadways

Roads should be built to provide access to crew change locations, inspection along by-pass yard tracks and easy access to other terminal facilities. They should be preferably hard surfaced, low maintenance roads and include the necessary clearances and signage around crossings and adjacent to tracks for safe vehicle movement.

2.6.5.6 Lighting

Adequate lighting should be considered for bypass yard leads, crew change points, engine tracks or other locations where regular activity will occur.

4.2 DESIGN OF INTERMODAL FACILITIES

4.2.1 Introduction

Intermodal terminals are specialized freight terminals designed to efficiently transfer trailers and/or containers to and from rail cars. Trailer on flat car/container on flat car (TOFC/COFC) operations involve mounting trailers or containers on specially equipped flatcars. Double-stack intermodal operations involve placing one container on top of another on specially designed rail cars that support the containers at a height of about one foot above the top of the rails. Intermodal terminals integrate rail, highway and waterway transportation modes. For additional pertinent information on design of TOFC/COFC Facilities, refer to Bulletin 696, Proceedings Volume 85, May 1984, Pages 157-190.

4.2.1.1 General

Factors influencing the facility location and design are accessibility to major highways and water routes, and capacity and clearance capability of the serving rail lines. The location studies must consider the equipment type, the traffic volume, railroad operations, highway traffic patterns and central location with respect to market area. There must be a commitment from railroad management concerning the general area where an intermodal or TOFC/COFC terminal is desired.

4.2.1.1.1 Operational Concept

The design of an intermodal or TOFC/COFC terminal will be governed principally by the volume of intermodal traffic, the land available, storage requirements and the existing layout for expansion projects.

A waterfront terminal serving a port handling container ships usually requires more storage/parking area than an inland terminal unless the operations of the two facilities are closely coordinated. Waterfront terminals will frequently be called upon to handle high volumes of movements on peak days and stand idle the remainder of the week. Operations commonly handling perishable loads have different needs since electrical outlets and additional fuel supplies are required to keep the refrigerated units running.

Standards should be developed to permit efficient truck-trailer movement between the terminal gate and the parking areas. A computer program to help locate and retrieve trailers and containers will permit more efficient location assignments for incoming and outgoing trailers.

A centralized management information system will facilitate trailer handling, spotting, preblocking and all associated paper flow.

4.2.1.1.2 Rail Equipment Considerations

A variety of specialized rail equipment is used in the transport of trailers and containers. Trailer carrying railcars provide a supporting platform for the trailer wheels and a stanchion to support the trailer at the king pin. The stanchion latches the king pin to provide longitudinal restraint to the trailer. Some of these railcars have continuous platforms and short bridge plates to span the gap between coupled cars, thus allowing the trailers to be driven onto and across the cars. Lighter cars have platforms only at specific support points and require that all trailers be lifted on and off the cars.

Containers are carried on two basic types of cars. These are single-level cars similar to those used for trailers and double stack cars specifically designed to carry two containers stacked one on top of the other.

Single level cars have special fittings that support and secure the corner castings of standard containers. Two types of double-stack car designs are in use. Both carry the lower container in a "well" with the bottom of the container supported approximately one foot above the top of rail. Bulkhead cars restrain the top container from longitudinal and lateral movement with bulkhead guides at all four corners of the container, holding the lower one foot of the container in place. The weight of the container is sufficient to prevent the container from bouncing out of the guides. Double-stack cars without bulkheads require the use of twist-lock inter-box-connectors (IBC's) to secure the upper box to the lower box. The IBC provides both horizontal and vertical restraint for the upper box. All containers must be lifted on and off the rail cars using various types of lift equipment that is generally dedicated to use at a given intermodal terminal.

4.2.1.2 Site Selection Planning

Many of today's TOFC/COFC terminals are inadequate because they were originally designed to conform to an available site. This approach should be avoided if at all possible.

Layout and planning for the facility should include the following elements:

4.2.1.2.1 Environment

Chapter 13, Environmental Engineering, AREA Manual for Railway Engineering discusses environmental considerations in detail. Environmental factors to be considered include:

1. Air
2. Water
3. Noise and Lighting
4. Rainfall Runoff
5. Archaeological and Historical Sites
6. Housing Displacement
7. Proximity to Residential Areas
8. Wetlands
9. Floodplains
10. Difficult soils conditions

4.2.1.2.2 Economics

The ideal facility topography is relatively level with good cross drainage and stable foundation material. The site should allow a design that facilitates 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.

4.2.1.2.3 Traffic Volume

Projected traffic volumes and possible future volumes will influence layout and traffic circulation plans.

4.2.1.2.4 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. Space is often limited and expensive, which therefore leads to more mechanized storage and handling systems.

4.2.1.2.5 Standardization

Standardizing certain elements of a TOFC/COFC terminal is desirable. This will permit the future transfer of terminal equipment from one terminal to another.

4.2.1.2.6 Highway Access

Good highway access is essential to the proper TOFC/COFC terminal siting. Highway load restrictions and clearances must be considered.

4.2.1.2.7 Rail Access

The approach tracks should be free from rail traffic congestion and have the proper rail clearances. If the daily volume of an intermodal terminal exceeds the track capacity of that terminal, additional support yard trackage will be required to accommodate arriving and departing trains and additional car storage.

4.2.1.2.8 Zoning

Most governmental jurisdictions have zoning laws that govern constructing the facility. It is highly desirable to avoid sites near residential areas or sites that require rezoning, as this is often a lengthy process with limited prospects of successful results.

4.2.2 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 describes mechanical loading considerations, loading methods and yard design considerations.

There are three types of TOFC/COFC facilities... end, side and overhead loading and unloading. Each has different cycle times.

4.2.2.1 General

The approximate cycle time for each method during typical TOFC/COFC loading/unloading operations 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).

4.2.2.2 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.

4.2.2.2.1 Low Volume – Less than 100 Lifts/Day

Low volume terminals are characterized by infrequent train service. Some parking or yard space is necessary. In many cases, trailers can be ramped as they arrive.

4.2.2.2.2 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.

4.2.2.2.3 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. They are usually designed to use side-loaders or overhead cranes for loading and unloading.

When a trailer enters the terminal area, the trucker may be directed to leave the trailer in a specific parking area or deliver it to trackside for loading. Hostlers pick up trailers from assigned parking spaces and spot them at trackside before loading operations begin.

The rail unloading cycle works in a similar manner but in reverse.

4.2.2.3 End-Loading (Fixed or Portable Ramps)

Railroad cars are end-loaded by backing the tractor-trailer combination onto 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.

4.2.2.4 Side-Loading

Side-loading and unloading can be done by a forklift 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.

Side-loaders have poor weight distribution which increases both the subsurface density and paving thickness requirements, and thus, greatly increases construction costs.

Side-loader characteristics vary depending upon 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

4.2.2.5 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 runways to support the wheel loads, while a rail-mounted crane requires a firm foundation to support the crane rail.

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. Ground operations supporting container loading/unloading operations are more complex because the bogies or chassis must be brought trackside.

The characteristics of gantry cranes vary depending upon 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.

Straddle carriers (Van carrier) also provide overhead loading capabilities. Unlike the gantry crane, the straddle carrier provides both the lifting and transport functions. The straddle carrier picks up the trailer or container at its current place of rest by straddling it and lifting it. The straddle carrier then transports the unit to the end of the string of railcars to be loaded. The unit then travels over the railcars, straddling the track and car, carrying the trailer or container in an elevated position. Upon reaching the car to be loaded, the unit is lowered into position. The process is fully reversed for unloading.

The characteristics of straddle carriers vary depending upon the manufacturer. The following list displays key information regarding the major types of straddle carriers now in use:

1. Capacity: 50,000 to 100,000 lb.
2. Span (rubber-tired): 15-20 ft.
3. Transport to storage area: eliminates the need for an independent hostling vehicle.
4. Lift height: One trailer over trailer on flat car; one container over two containers on a double-stack car.
5. Turning radius: 35 ft. outside radius.
6. Number of lifts per day: Depends on travel distance to staging area.
7. Travel speed: 23 mph

The choice of equipment characteristics must be made in concert with other facility development issues, such as: track centers, traffic flows and operating volumes.

4.2.2.6 Parking/Staging Equipment

4.2.2.6.1 Containers

Yard tractors include flatbed trucks and trailers/chassis units which move containers from trackside to the parking area.

Straddle-carriers are specialized units designed to transport one container at a time between trackside and the parking area.

Heavy-duty forklift trucks are used for stacking and repositioning containers.

Travel cranes are mounted on either rubber-tired wheels with straddle widths up to 75 ft. or rail mounted with straddle widths up to 200 ft.

4.2.2.6.2 Trailers

Trailers are usually moved between trackside and the parking area by a yard or road tractor.

4.2.2.7 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 presently used for TOFC/COFC service. However, the trend is toward longer cars carrying two of the longer highway trailers on a flat car.

4.2.2.8 Special Intermodal Cars

Various kinds of intermodal cars have been designed to reduce weight, improve aerodynamic efficiency, improve train handling characteristics to reduce damage to lading, reduce fuel consumption, reduce the number of locomotive units needed to move a given consist and improve terminal operation. Several types of cars now in common use include:

1. Single Platform Skeleton Cars. These cars are designed to carry containers or trailers on a lightly framed car. These cars are generally equipped with special single axle trucks at either end of the car. Each car is capable of carrying one trailer or one long or two short containers.
2. Articulated Skeleton Cars. These cars are generally made up of five independent platforms similar to the single platform cars. The five platforms are carried on a total of six two axle trucks. Each platform is capable of carrying one trailer or one long or two short containers.
3. Single Platform Double-Stack Well Cars. These cars carry one or two containers in the well and one more container on top of those in the well. The top container is secured to the lower container(s) with inter-box-connectors (twist locks).
4. Articulated Double-Stack Well Cars. These cars consist of five well car type platforms connected with articulated couplings, and carried on six two axle trucks.
5. Articulated Double-Stack Bulkhead Car. These cars are very similar to the well type car except that the upper container is held in place by fixed or adjustable bulkheads located at each end of each platform.
6. Articulated well-cars, some with a mix of trailer and container carrying capabilities.
7. Stand-Alone Well Cars. These cars are basically a series of single platform well cars connected with rigid drawbars in order to achieve a train with a minimum of slack action.
8. Dual-mode vehicles (rail/highway vehicle). These vehicles are specially designed highway trailers that either have a single rail axle permanently mounted to the trailer, or connect to a special two axle rail boggie. The individual units are connected to each other in elephant train fashion. A special connector on the nose of one trailer is used to connect that trailer to a receiving socket at the back of the preceding trailer. A large pin locks the two units together.

4.2.2.9 Trailers

The size and weight of truck trailers operating over highways are controlled by state and federal law. Federal Law permits 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 53 ft. long on sections of the federal aid primary system highways. The allowable load limits and the seasonal weight restrictions on the access roads to the TOFC/COFC terminal are important.

4.2.2.10 Containers

Containers come in a variety of common sizes, however their fittings and securement devices are usually standardized based on the location of these devices on 20 foot and 40 foot ISO (International Standards Organization) marine containers. Common container lengths include 20′, 40′, 45′, 48′ and 53.′ 24′ and 35′ containers are found in the fleets of some steamship lines. Introduction of a 28′ unit is anticipated. Widths are generally 96″ or 102″ with heights being 4′, 8′, 8′-6″ and 9′-6″.

4.2.2.11 Securement

Trailers are secured to the railcar with a stanchion support that locks onto the trailers kingpin. These stanchions may be a fixed type if the car is only designed for TOFC service. The stanchions are retractable if they also accommodate container service or if they are designed for loading by the "End Loading" method. (4.2.3.1.1) Two types of retractable tie-down mechanisms or trailer hitches on flatcars are presently in use, the "wrench-operated" and "tractor-operated." Trailer wheels are not secured, but lateral movement is resisted by curbs in the wheel support area.

Containers are secured in a variety of ways. On flat cars or most skeleton cars, the container is supported on pedestals at all corners. The pedestal provides vertical support plus lateral and longitudinal restraint. A special spring clip provides vertical restraint. Double-stack container cars restrain lateral and longitudinal motion of the lower container with fixed guide pins on the support plate. These pins mate with a standard casting on the container at the 40′ x 96″ location. The container sits deep within the frame of the car so that lift out is not a problem. For well cars, the upper container is locked to the lower container using standard marine type interbox-connectors at the 40′ x 96″ location. On bulkhead cars the upper container is also supported on the lower container, but all other restraint is provided by the bulkheads. A special saddle is being proposed to carry two 28′ containers in the upper position on double-stack well cars.

4.2.2.12 Chassis

Provisions for chassis storage should be made in the design for all terminals that will handle containers.

4.2.3 Design Factors

The design factors that must be considered include the type of terminal, layout and configuration, pavement types, parking and storage, security, facility services, environmental controls, the terminal buildings and the maintenance and service buildings.

4.2.3.1 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 or at an assigned trackside position for subsequent loading.

4.2.3.1.1 End Loading

The ramps for stub-end tracks can be constructed from timber, steel, or concrete filled with earth. Tracks 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. Between-track platforms or platform walkways adjacent to the tracks are desirable. These platforms, which permit easy worker movements between cars, should be about 2 ft. 3 in. wide and 3 ft. 6 in. high or car floor-height. Laws governing track clearances affect the width of these platforms.

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.

4.2.3.1.2 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. The single track facility can be readily expanded as shown in Figs. 3 & 6. 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 stall markings and signing.

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.

Side loading of containers requires the coordinated efforts of the equipment operator and several truck or hostler drivers as the container chassis must be removed from trackside to allow the equipment to approach the railcar.

4.2.3.1.3 Overhead Loading

Replacing side-loader equipment with crane-loading equipment should be explored when lift volumes approach 250 to 350 lifts per day. 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. Figs. 7 & 8 show a configuration for loading double stack rail equipment using long span and short span overhead equipment.

Terminal operators will vary in their opinion regarding the merits of sidelift equipment versus overhead lift equipment. This is generally a speed vs. flexibility argument. The overhead equipment has faster cycle times and is very efficient when moving from one end of the track to the other loading or unloading a unit at each position. Side loading equipment generally has a higher ground travel speed allowing it to move around the facility quicker to handle "Hot" loads at random locations.

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 located at strategic points around the city should be contrasted with the efficiency of a single very-high-volume terminal.

4.2.3.1.4 Lift and Travel Loading

Straddle (van) carriers provide a variation on the other mechanical systems in that the lift and travel functions are provided by a single piece of equipment. This approach allows a terminal to expand through a range of throughput rates without altering the basic organization of the facility.

4.2.3.2 Layout and Configuration

The type of loading-unloading equipment to be used in a terminal influences the terminal layout and configuration.

Terminal layouts also affect the efficiency of loading/unloading and parking activities. Rail-mounted cranes require the least amount of lateral space but may complicate the problem of movement between rail cars. Rail-mounted machines are used where they can serve several tracks from a single travel path. While portable ramps are inexpensive, access room is required at the end of the rail cut. Side-loaders are flexible, but need adequate operating space between parallel tracks. Rubber-tired, overhead-lift equipment can function in a wide range of layout widths, but when configured for the maximum operating flexibility has the widest site requirement per track. Van carrier systems require the least site width per track of all the mechanized systems, but always require an independent parking area. The parking area for a van carrier facility requires more land per parking stall than other systems because of pathways required for the carriers wheels. The requirements of a combination TOFC/COFC facility should be considered when determining the most effective equipment and layout.

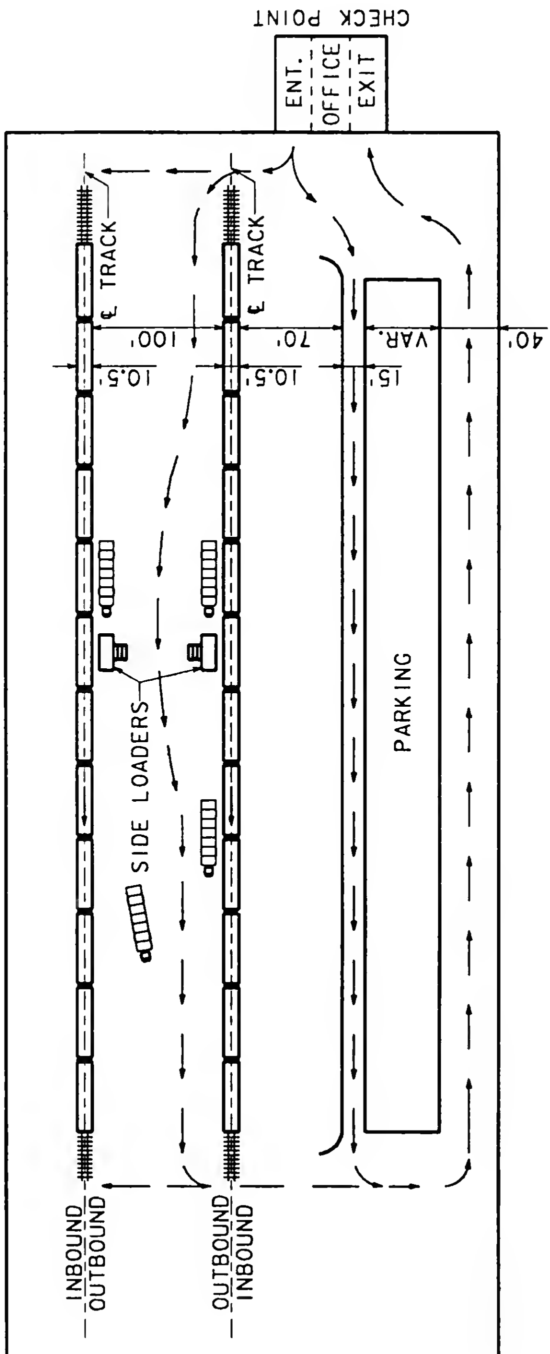


Figure 6. Side Loading Double Stack Cars Between Parallel Tracks

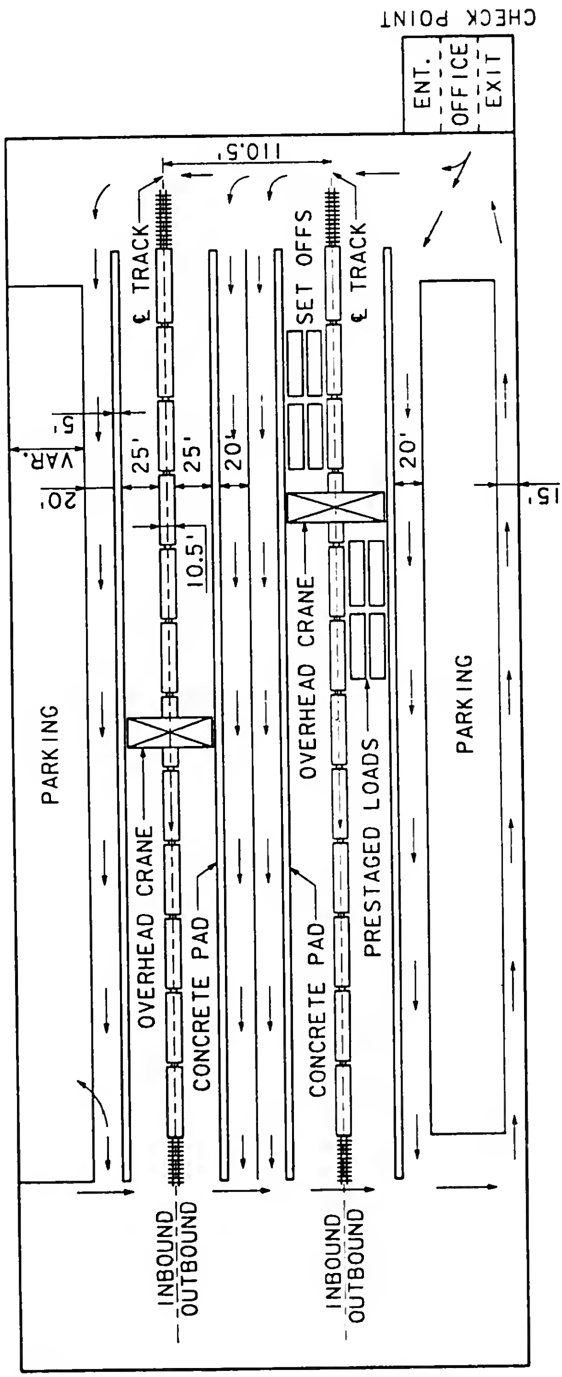


Figure 7. Long Span Crane, Single Track With Double Stack Cars

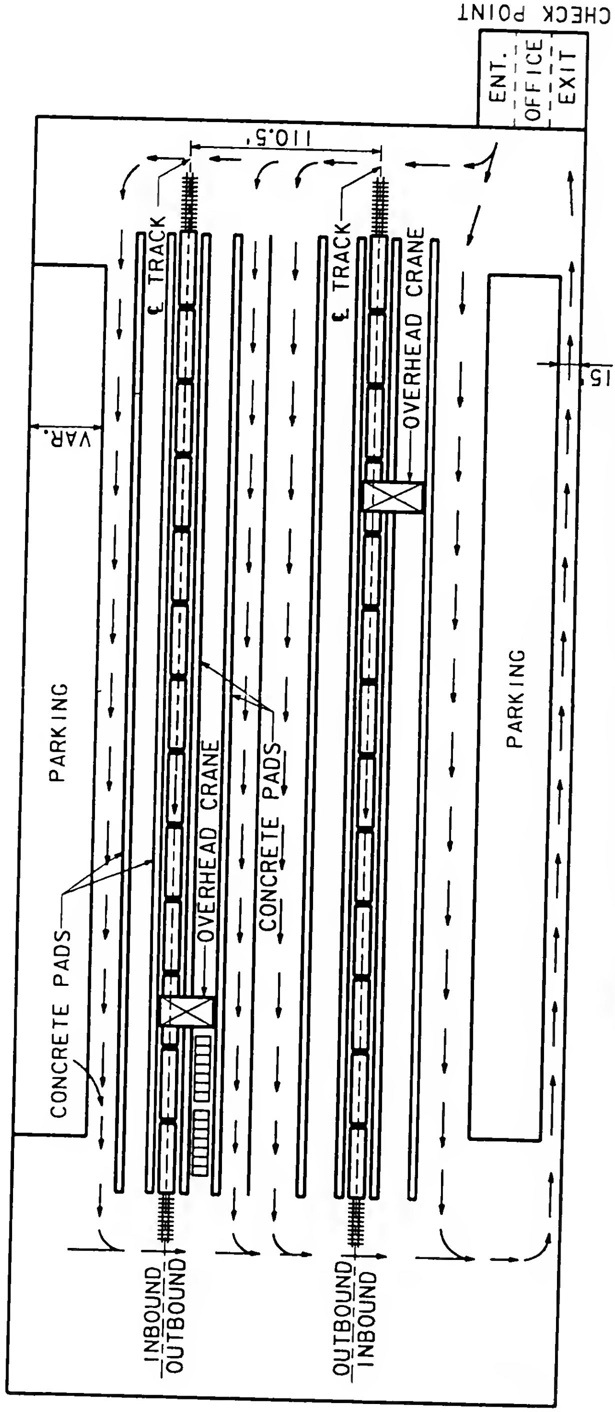


Figure 8. Short Span Crane, Single Track With Double Stack Cars

Medium-volume terminals consist 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 have typical track lengths of 3,000 to 8,000 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.

Curves on the approach track should have the longest radii possible, with a minimum allowable radius on a constant vertical gradient of 441.68 ft. (13°). Due to extreme length of rolling stock, design of minimum radius may result in operating difficulties.

4.2.3.3 Pavement Systems

4.2.3.3.1 General Considerations

There are three primary requirements for pavement at TOFC/COFC terminals.

1. The capacity to support parked loaded semi-trailers and containers;
2. 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,
3. 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.

4.2.3.3.2 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 and their characteristics.

The maximum grades recommended for a semi-trailer storage area are from 0.5 to 1 percent.

Access roads should have a minimum of two 12-ft. lanes with a minimum 5-ft. shoulder on each side.

4.2.3.3.3 Portland Cement Concrete Wearing Surface

Runway widths vary from 5 to 10 ft. Thickness will depend on factors given in the references mentioned above in paragraph 4.2.3.3.2.

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.

4.2.3.3.4 Asphalt-Concrete Wearing Surface

This paving is frequently used for trailer/container parking space and for truck driveway and maneuvering areas.

4.2.3.3.5 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.

4.2.3.3.6 Waterbound Macadam

This is the least expensive type of paving and can give satisfactory service at small-to-medium-size ramp-loading terminals.

4.2.3.3.7 Specifications and Construction Procedure

Individual state highway department standard specifications can be used for constructing the pavement systems or airport pavement criteria can be used for the design of crane pavements.

4.2.3.4 Parking

Parking facilities should be near the loading tracks with additional parking for storage, as required.

4.2.3.4.1 Trailers

Yard and road tractors are the primary method of moving trailers within a terminal. The trailer parking configuration shown in Fig. 1 through 5 should be adequate to accommodate the daily traffic in each type yard. A 10-by-50-ft. parking area should be provided as a minimum for each trailer. Additional length may be desirable to avoid having 53 ft. units encroach upon aisleways. Wider slots may make maneuvering easier but does so at a significant loss in the total number of parking slots that can be provided on a given piece of land.

A trailer parking area of approximately two-and-one-half times the number of trailers handled each day should be planned. This ratio is based on historical trailer dwell periods experienced at intermodal terminals. The amount of parking necessary may vary widely for terminals that primarily serve a single customer, such as a marine terminal, the U.S. Postal Service or UPS.

4.2.3.4.2 Containers

A 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. 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.

There are three basic configurations for parking/stacking the containers.

1. Herringbone Layouts–used to store containers either on chassis or on support legs, with no stacking.
2. Block Layouts–the best use of scarce parking areas; containers are stacked three or four high in a tight block. Block layouts are often used for storing empty containers and for long-term storage of containers awaiting outbound movements.
3. Ribbon Layouts–offer better container selectivity than block layouts.

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.

4.2.3.5 Security

TOFC/COFC facilities are easy targets for both organized and random burglary. Further, the terminals are often in high-crime environments where theft operations can be highly organized. Security for TOFC/COFC facilities is therefore essential. Security measures include fencing, lighting, guards and patrols, closed circuit TV to scan the terminal and sensor systems.

4.2.3.6 Facility Services

4.2.3.6.1 Electrical

Lighting and power outlets in the track area may be provided to assist tie-down operations. Parking areas should be lighted if there are extensive night operations or for security and safety reasons.

Typical design procedures and criteria are published in the Illuminating Engineering Society's IES Lighting Handbook.

4.2.3.6.2 Communications

Communication facilities within and beyond the operation area should be provided for efficiency. In addition to traditional phone systems and two-way radios, modern communications systems include computer-in-cab systems for both cranes and yard hostlers, radio frequency automatic equipment identification systems, and localized geopositioning systems.

4.2.3.6.3 Utilities

Underground utilities are desirable to avoid conflict with operation. Sanitary, water, HVAC, electrical utilities and possibly engine block heaters should be provided in accordance with the facility requirements. Fire Protection should be provided as stipulated in Chapter 14, Part 1, Section 1.6 of the AREA Manual.

4.2.3.6.4 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. Other methods use trench drains between the track and pavement sections, or slotted drains in large paved areas.

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.

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 systems.

Care should be taken with the design of drainage systems where containers are staged or stored in grounded stacks. Where possible drainage should be directed away from these areas. Other methods include providing slightly raised concrete pads to support containers on their corner castings, allowing a few inches of space under the body of the container for the passage of surface water.

4.2.3.6.5 Water Pollution Control

Water pollution control ranging from oil/water separators to full treatment and pH balancing may be required at the following service areas: (Also see Chapter 13)

1. Fuel
2. Maintenance Building
3. Outside Maintenance Areas
4. Trailer and Truck Washing Facilities
5. Paved Parking Areas

4.2.3.6.6 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. See Chapter 34 of the AREA Manual for details.

4.2.3.7 Terminal Buildings

Complete design criteria available in AREA Manual, Chapter 6. However, for information note the following:

4.2.3.7.1 Offices

The larger-volume operations will require an office for supervisory and clerical staff, with the normal amenities for operating personnel. Standard office design criteria should be used, including provisions for communications, uninterruptable power supply and employee parking.

4.2.3.7.2 Storage Building

A storage building should be provided for blocking and bracing material for adjusting shifted loads.

4.2.3.7.3 Air Compressor Facilities

Air compressor facilities are required for making brake tests on cars and for the use of air tools.

4.2.3.7.4 Interior Washing Facilities

Interior washing facilities and appurtenances may be necessary if refrigerator trailers are handled in sufficient quantity.

4.2.3.7.5 Container Transloading Building

A container transloading building may be required.

4.2.3.7.6 Guard Building

When the office is not located at the entrance, a separate guard building should be provided for check-in and check-out and equipment condition inspection. If not provided elsewhere, the guard building may also be used to support security activities and provide for outside communication.

There are several methods of making trailer/container roof inspections when required.

1. Overhead mirrors;
2. High platforms with ladders or stairways; and
3. TV cameras monitoring from the interior of the office building.

4.2.3.7.7 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.

4.2.3.7.8 Gate Check-in Facilities

High volume facilities require efficiently functioning gates to maintain a smooth flow of vehicles into and out of the terminal. Special lanes may be provided for bob-tail tractors. Telephone pre-check stations allow drivers to give clerks all necessary information before they proceed to scales and inspection stations. By executing a pre-check, an interchange document can be printed at the inspection station before the driver arrives.

4.2.3.7.9 Hazardous Material Containment

A special area of the terminal may need to be set aside where hazardous material can be contained if a leaking container or trailer is encountered.

4.2.3.8 Maintenance and Service Buildings and Facilities

4.2.3.8.1 Locomotive and Car Maintenance

Maintenance operations for locomotives and cars at TOFC/COFC facilities are usually done at nearby service 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, however with the intense use of double-stack container cars, heavier repairs such as wheel changeouts are also undertaken while the car is in the terminal rather than sending it to a shop location for this work.

4.2.3.8.2 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. In the case of refrigerated units, diesel-powered generators must be checked. Tractor maintenance and service may be provided when necessary.

Other repairs that may be accommodated at an intermodal facility include:

1. Watertightness repairs to trailers and containers.
2. Tire repairs to hostlers, trailers and chassis.
3. Lamp and lens replacement for chassis and trailers.

4.2.3.8.3 Equipment Fueling Facility

Fueling facilities for equipment should be considered. The equipment requiring fueling facilities are:

1. Tractors;
2. Refrigerated Trailers;
3. Gantry Cranes;
4. Side Loaders;
5. Portable Generators;
6. Straddle Cranes; and
7. Other Mechanical Equipment

4.2.3.8.4 Side-Loader/Crane Maintenance Facility

A separate building will improve maintenance of this equipment, especially in colder climates.

Complete design criteria and additional pertinent information are readily available in Bulletin 696, Proceedings Volume 85, May 1984, pages 157-190.

5.2 SERVICING FACILITIES

5.2.1 Fueling Stations

5.2.1.1 General

At locations where locomotives are to be fueled, facilities must be provided for receiving, storing and dispensing the fuel, unless fueling is to be direct from tank truck to locomotive.

In the design and construction of fueling stations provisions should be included to prevent the pollution and contamination of public waters from spilled fuels through surface and subsurface waters, sewers and other conduits. For environmental design considerations refer to Chapter 13, Part 5, Section 5.6, FUEL AND LUBRICATING OIL SYSTEMS.

Diesel fuel may be delivered to a fueling station by rail tank car, truck, pipeline or boat, if fueling location is on or near navigable waterway.

Fueling may be performed at stationary facilities to which locomotives are moved, or by mobile servicing units which go to the locomotives.

5.2.1.2 Location

Stationary fueling facilities should be located at locomotive terminals in line with other servicing facilities, where provided.

If run-through trains are to be fueled, consideration should be given to locating fueling facility on main line or thoroughfare track at crew change point.

5.2.1.3 Tracks

Track(s) provided for fueling should have capacity equal to the largest diesel consist which is to be serviced at the facility.

Where diesel fuel is to be received by rail tank car a separate unloading track should be provided with sufficient capacity for the largest fuel shipment to be handled.

A thoroughfare track should be provided to transfer locomotives to and from the fueling station.

Tracks on which fuel is to be unloaded or dispensed should have gradients as flat as possible, preferably but not exceeding 0.1%. Portions of tracks wher cars are to be spotted for unloading and where locomotives will be fueled should have tangent alignment. Curvature should not exceed 12 degrees on any fueling station tracks.

A blue flag system should be provided on loading and servicing tracks.

5.2.3 SANDING

Sanding facilities should be provided to serve all locomotives entering and leaving the terminal.

Sanding may be performed at stationary facilities to which locomotives are moved or by mobile facilities which go to the locomotives. For design considerations for fixed facilities to unload, store and load sand into locomotives refer to Chapter 6, Part 6, LOCOMOTIVE SANDING FACILITIES.

5.2.3.2 Location

Sanding facilities are usually situated adjacent to, and in line with, fuel and water facilities so that locomotives can be completely serviced at one location.

5.2.3.3 Tracks

A sanding track should be provided with capacity not less than the largest locomotive consist normally operating in, or through, the terminal.

If locomotive sand is to be received in covered hoppers or other rail cars, a separate unloading track should be provided with sufficient capacity to hold the largest shipment.

The portions of track on which sand is to be loaded into locomotives or on which sand is unloaded into storage should have the flattest gradient possible, preferably not exceeding 0.1%. Alignment of track through loading and unloading stations should be tangent. Curvature on other portions of the track should not exceed 12 degrees.

Blue flag protection should be provided at loading and unloading stations.

Proposed 1993 Manual Revisions To Chapter 27 - Maintenance of Way Work Equipment

It is proposed to revise paint specifications in Part 1 - General Recommended Colors for Painting Motorcars, Roadway Machines and Work Equipment, and Part 2 - Roadway Machines, Specifications for On-Track Roadway Machines, Section 17 - Paint. The new paint specification will specifically identify color shades for painting various areas or components on Roadway Work Equipment, utilizing shades selected from Federal Standard 595-B, Colors Used in Government Procurement. The proposed replacement specifications for Parts 1 and 2 are as follows:

Part 1

General

AREA Yellow

[1]Recommended Colors for Painting Motor-Cars Roadway Machines and Work Equipment

The most suitable colors for painting Motor Cars, Roadway Equipment and Work Equipment are as follows:

Equipment	Color	Standard Spec. No.
Motor Cars	Yellow	13538
Roadway Equipment	Yellow	13538
Roadway Work Equipment (cranes, etc.)	Yellow	13538
Work Equipment (Jordan Spreaders, etc.)	Mineral Red	10076

The predominant colors of equipment will be a specified color, but some parts or components may require custom painting (reference AREA Committee 27 - Specifications for On-Track Roadway Equipment Section 17 - Paint). Some of the more notable items are listed below:

Component Area	Color	Standard Spec. No.
Gasoline tank	Red	11086
Diesel Fuel tank	Green	14110
Hydraulic Reservoir	Blue	15180
Coolant Reservoir	Gray	16251
Lifting Lugs	Black	17038
Safety Locks	Red	11086

The colors listed under the Standard Specification Numbers are consistent with those indicated in the "Federal Standard 595-B, Colors Used in Government Procurement."

NOTE: It should be understood that some Railroads may have special painting requirements which will supersede the above recommended practices or guidelines.

Specifications for On-Track Roadway Machines. Revision of Section 17 - Paint.

17. PAINT

Manufacturer shall use paints which do not require periodic medical examinations or special equipment, other than passive respirators. It is assumed that no-lead paint with a lower percentage of solvent may be required and that exact colors formerly furnished may not be available. Manufacturer shall provide the colors listed under the 'Standard Specification Number' which are consistent with those indicated in the "Federal Standard 595-B, Colors Used in Government Procurement."

Manufacturer shall also utilize proper surface preparation, including primer, to provide a high quality, durable finish coat. General exposed parts of the machine shall be painted AREA Yellow, Spec. No., 13538, unless otherwise specified by a Railroad, EXCEPT AS FOLLOWS:

Equipment & Component Area	**Color**	**Standard Spec. No.**
Gasoline tank*	Red	11086
Diesel fuel tank*	Green	14110
Hydraulic reservoir*	Blue	15180
Coolant tank*	Gray	16251
Wheels and handrails	Black	17038
Lifting lugs	Black	17038
Jacking points or pads	Black	17038
Safety locks	Red	11086
Engine & other Misc. parts	Manufacturer's option	

***NOTE:** Where side or top of tank containing filler opening has an area greater than 144 square inches, manufacturer may elect to paint only 144 or more square inches at the filler opening with the required tank color. Name of fluid and the words "CHECK DAILY" shall be stenciled on each tank in 1-inch letters. Total machine weight with all tanks filled shall be plainly marked on both sides of the machine in 1-1/2 inch letters:

Weight ________ lbs.

Comply: Yes ________ No ________ Remarks: ____________________

Proposed 1993
Portfolio of Trackwork Plans Revisions

The following 14 plans are considered obsolete and are proposed to be deleted from the Portfolio of Trackwork Plans. This includes the 600 series light wall rail bound manganese steel frogs, Plan Nos. 600, 612, 613, 614 and 615, for No. 6 to No. 20 designs. Specifically the plans are:

Plan No. 255-34
Plans and Specifications for Switch Lock

Plan No. 505-59
Specifications for Guard Rail Clamps

Plan No. 510-40
Manganese Steel One Piece Guard Rail

Plan No. 600-80
Data and Sections for Rail Bound Manganese Steel Frogs

Plan No. 600-A79
Design Criteria for Rail Bound Manganese Steel Frogs

Plan No. 612-79, No. 6, No. 7 and No. 8
Rail Bound Manganese Steel Frogs

Plan No. 613-79, No. 9, No. 10 and No. 11
Rail Bound Manganese Steel Frogs

Plan No. 614-79, No. 12, No. 14 and No. 15
Rail Bound Manganese Steel Frogs

Plan No. 615-79, No. 16, No. 18 and No. 20
Rail Bound Manganese Steel Frogs

Plan No. 721-34
Design of Reinforced Concrete and Pile Crossing Foundations

Plan No. 782-80
Articulated Manganese Steel Crossings, Angles 90° to 60°, incl.

Plan No. 783-80
Articulated Manganese Steel Crossings, Angles Below 60° to 40°, incl.

Plan No. 913-52
Approach Ties for Open Deck Bridges and Trestles

Plan No. 1004-52
Data for Headfree Rail Sections

NOTES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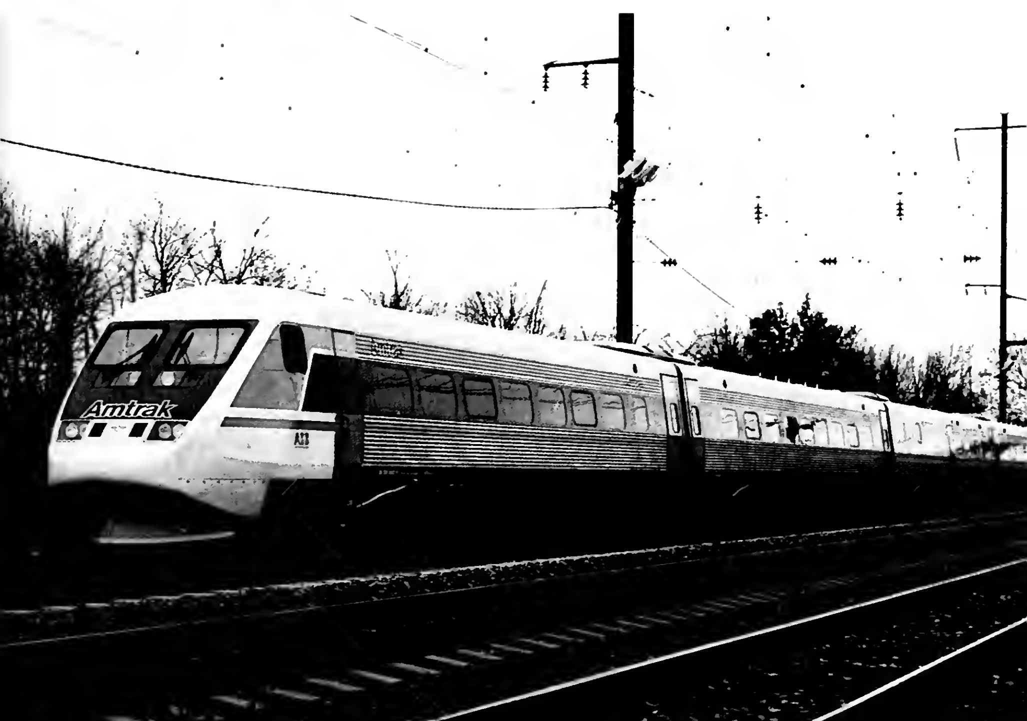

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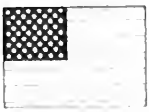

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Stefanie Streever, Editor

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135 M.P.H. REVENUE SERVICE ON AMTRAK

In mid-October, Amtrak acquired an X2000 tilt-body trainset on loan from the Swedish State Railway. The train was brought into the U.S. to test and demonstrate the feasibility of this technology in this country.

The X2000 is a five car trainset, hauled by a 4300 horsepower electric locomotive. The cars can tilt which allows the train to negotiate curves at speeds 30 to 50% higher than our trains. The suspension, which allows such performance, is radically different from that found on conventional trains; the axles are permitted to move within the truck frame and conform to the shape of the curve. Instead of remaining parallel, the axles are allowed to take a radial position and point to the center of the curve.

In late fall and early winter, the trainset was tested on selected curves on the Philadelphia to Harrisburg, and the Washington to New York main lines. With conditional approval and supervision of the Federal Railroad Administration, curving tests were conducted at up to 12 inches of cant deficiency. The tests were performed with a full complement of instrumentation, including four sets of instrumented wheelsets. These wheelsets gathered data on lateral and vertical wheel forces, truck lateral forces, as well as L/V ratios. The tests demonstrated that the X2000 can safely operate at curving speeds producing up to the maximum tested value of 12 inches.

Also, during this time, the train was operated up to 155 mph on Amtrak's existing main line. The trainset showed no signs of instability at this speed.

As a result of these tests, the FRA has approved Amtrak's request to operate the train in scheduled daily revenue service at 135 mph and at 9 inches of cant deficiency.

(for details, see article on page 141)

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PRESIDENT'S ADDRESS

By: W. E. Glavin*

Ladies and Gentlemen, it is with great honor that I address this body as its president. The privilege to serve this past year has truly been a rewarding and unforgettable experience and the high point of my career as a railroad engineer. It is especially gratifying considering that I am third generation railroad engineer, with one grandfather in Work Equipment, the other in Bridge Design, my grandmother who was a Roadmaster's clerk, and my father a Chief Engineer and Vice President on the Pennsylvania, Penn Central, and Grand Trunk Western as well as president of this association just 11 years ago. I guess the good news is that you won't have another Glavin as president for at least another 20 years, since my two daughters are less than 5 years old and Mary isn't interested in pursuing a career in Railroad engineering, though I think she could do a bang up job.

This past year saw many accomplishments for the Association as well as some new directions. In spite of a less than stellar national economy and downsizing efforts at most companies represented here, overall membership remains relatively untouched. During 1992, we recognized the growth occurring in the transit field and secured a board position for a transit representative. This constitutional change was overwhelmingly approved by the membership. The new transit representative will be announced at tomorrow's luncheon, but he has already sat through his first board meeting.

We also completed a review of the operations and organization of the Association through the use of an outside consultant. In the ever changing landscape of business and management philosophies it is important that we look inward to be sure that we are making the best use of our resources and providing the greatest value to the membership and the companies for which they work. The consultant's overall report was extremely favorable, praising the mission and the direction of the Association, its value to its members, the level of control exerted upon the committees to ensure timely and relevant topics, and the list goes on. They also made some minor recommendations regarding coordination of committees, and the consolidation of some. For the most part, the Ad Hoc review committee led by Rich Johnson and the board of direction recognized these changes for what they were, merely cosmetic, and didn't progress them, in spite of the fact that we tried incorporating them into the fall chairman's meeting in Mexico.

The Ad Hoc committee and the Board also recognized that real or perceived duplication of efforts between the AREA and other industry associations could be deadly in this day and age of cost consciousness. This led to the consultant-recommended consolidation of Committee 22 into 16 as well as board recommended consolidations of Committee 9 into 5, and 13 into other related committees. The most controversial change has been the consolidation of the environmental Committee 13. Its consolidation has been perceived, by some, as a lack of recognition of the importance of environmental considerations in the industry. Nothing could be further from the truth. We recognized the duplication of the AAR function which already handles environmental issues as well as the fact that the AAR, with its budgets and presence in Washington, was far better suited to progress the railroad environmental cause than we are. Implicit in that recognition were certain assumptions regarding the AAR taking the responsibility for those functions that were unique to the AREA and its membership. The AAR has assured us that this will be the case. However, as with all assumptions, these need to be revisited in the future to make sure that all of the possible synergies were gained and that no aspects fell through the cracks. Indeed, if the assumptions are proven wrong in time, I would expect that the board in place at that time would adopt the establishment of a new committee to readdress those issues.

I have enjoyed my term as president and feel that I experienced personal growth as well. The job is not an easy one, as you have heard before. Not only are you doing your regular job back at home, but you also assume the responsibility for a lot of extra hours dedicated to leading the association. This is especially difficult when you get promoted to a new position in which you have no background as I was in the middle of my term.

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*President, American Railway Engineering Association, 1992-1993; Vice President Equipment Management, Burlington Northern

I want to thank the Headquarter's staff for seeing me through that time and allowing me to put my role on automatic pilot for a while. Wendy Taymen and Lou Cerny did yeoman's service to keep it all going. Beyond that, Stefanie Streever deserves special mention, because she did both her and Tom Smithberger's jobs, including assembling the conference program, after Tom left for greener pastures. Please join me in recognizing those efforts.

I also want to thank my wife Mary for putting up with the hours and the absence from home in order to conduct meetings. Without her undying support and explanations to my daughters Amy and Laurie, who are both here at the conference, I would not have been able to make it. By the way, Mary has put together a fairly untraditional spouses program that will be enjoyable.

Railroad Engineering is both an art and a science. The technicians in the trenches performing analysis and design are the scientists of the effort. Those that manage and direct those efforts, manage people, develop concepts and ideas, are the artists. This association is made up of both. As such, in our daily jobs we have to be tied to the past, the present and the future. One can't lose sight of the basics in developing plans for the future, because that is what railroad operations are firmly rooted in. At the same time, one can't be so entrenched in the past that one can't imagine the possibilities of how things could be if... Change is hard, change is resisted, change is painful. However, it is through change that we grow and improve. I've been fortunate in my career that I've been encouraged to innovate and solve old problems in new ways. In essence, my philosophy is to approach things from a nontraditional angle; break down issues to their most basic elements; alter and change each one if possible and rebuild from there. I hope that my tenure in this association has demonstrated the power of this process and the need for rapid and progressive change, regardless of the pain, because the greatest legacy I can leave is "if it isn't broke, make it better." Tinker, and you might find out something; don't do something just because it is the way we've always done it or because "the old ways are the best." I'd like nothing better than to hear the words, "Welcome to the AREA and the world of railway engineering. Please set your watches ahead 10 years." I appreciate the support I've received in those efforts from the trade press, the suppliers, the Burlington Northern Railroad, the board of direction, and the members of this association.

I understand that my reputation has preceded me on my new job and some car manufacturers are already quaking in their boots. I look forward to dragging them kicking and screaming into the 21st century.

Again, thanks for your attendance at these very important sessions. I hope that you enjoy the conference and learn from the information presented and return to your job more knowledgeable and as a more valuable asset to your employer or clients.

AMTRAK'S X2000 TESTS

By: E. J. Lombardi* and A. E. Shaw, Jr.**

The X2000, for those of you who are not familiar with the train, was brought to this country, by Amtrak, to test and demonstrate the suitability of many features. Primary among them, is the ability of the cars to tilt around curves, allowing the train to operate with shorter trip times than our normal trains. This will allow us to better compete in the New York to Washington and the New York to Boston corridors.

The train is on a short term lease from the Swedish State Railways, (SJ), and will be returned in August.

The X2000 trainset was designed and built in Sweden, by ABB (Asea, Brown-Bovari) and was a mutual development project between ABB and SJ. In the mid- to late-70s, SJ was losing ridership between Sweden's two largest cities, Stockholm and Gothenburg. Potential passengers were using the airline, rather than the train, because of the shorter travel time. The train took four hours to make the 285 mile trip, compared to about two and a half to three hours for the plane.

To stem this flow of passengers, and to improve their competitive position, SJ looked at various methods to shorten the travel time from four hours to three. One approach was to use the French method and build a new, straight line railroad between the two cities, upgrade the signal and catenary system, and operate trains at very high speeds. The second option was to operate on the existing lines with trains that could maintain a high average speed by tilting around curves. Their existing track was in generally good condition, but had a lot of curves and was signaled for approximately 80 mph.

Of the two approaches, the use of a new, tilting train capable of speeds of 125 mph was the most cost effective. Over the next decade, the line speed was upgraded, and a number of prototype vehicles were tested, ultimately, the X2000 was designed and built. The first trainset was placed into revenue service, by SJ, in September, 1990.

The original order was for twenty five-car trainsets. At this time, fifteen have been delivered. (Trainset fourteen was the one shipped to this country.) Recently, SJ exercised an option to purchase 15 more trains, for a total of 35, to cover additional X2000 routes to be added.

The soon-to-be Amtrak X2000 locomotive was modified in Vasteras, Sweden. A new main transformer was added because of the difference between their overhead catenary voltage and frequency. The control system and air brakes were also modified. Since their pantographs did not have the high reach that we require, two Amtrak pantographs were sent to Sweden to be installed. Of course, all the meters and gauges were changed to English as were most of the printed labels and instructions. There were a number of other modifications made to allow the train to operate in this country, including the addition of an Amtrak cab signal and speed control system.

In mid-October, the train arrived in Baltimore. Each vehicle was tied down on its own dolly, or mofi. The yellow wheels are instrumented wheelsets; they were installed in Sweden for the high cant deficiency tests to be performed in the United States. Two were installed under the lead truck of the locomotive and two were under the lead truck of the cab car, which is the last coach of the train. The cab car is a coach equipped with a locomotive cab for dual direction operation.

The X2000's attraction to Amtrak is its ability to operate safely at high speeds around curves, and to tilt, thus allowing for a comfortable ride. Although the X2000 has operated for many years at high curving speeds, first as a prototype, then as a production model, the FRA needed to see track and rail force data generated in Amtrak's track. The high cant deficiency tests will be discussed next.

Thirty channels were constantly displayed for immediate viewing and analysis of the tests. Although a wheelset computer was also analyzing the same data at the same time, the actual data was displayed in

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*Manager—Performance and Tests, Amtrak
**Director—Engineering Tests and Standards, Amtrak

---

real time on these strip chart recorders so that we could make an instant decision to stop the testing, should it become necessary. All of the instrumented wheelset related hardware and software was designed, owned and operated by SJ.

The vertical and lateral force data from all eight instrumented wheels (four wheelsets) was displayed in real time. Data was computed and displayed for axle lateral, total truck-side lateral, and total truck-side vertical. In addition to the wheelset data, there were a number of truck frame and axle accelerations that were displayed.

In addition to the thirty channels of on-line monitoring, other channels were also acquired and stored for later, off-line analysis.

Before the X2000 was brought into this counrty, Amtrak, along with the FRA made a preliminary investigation of the wheel/rail forces. We wanted to ensure that the X2000 could operate safely on our infrastructure.

The initial investigation was with ABB and SJ. With their full cooperation, we looked at the results of previous instrumented wheel tests, performed on SJ, we were specifically interested in the forces that were imparted into SJ's track structure. Although their track structure is not identical to ours, this study was a place to start our investigation.

Secondly, ABB was asked to compare the predictions of their mathematical simulation of the X2000's suspension with actual test results. These comparisons were very favorable and gave us a high degree of confidence.

Third, we provided ABB with geometry data for three curves on our Harrisburg line. This data, on magnetic tape, was used as input to the X2000 model and indicated that there would be no safety-related problems at up to at least 14 inches of cant deficiency. Since we planned to test only to 12 inches, we had a high degree of confidence that we had a vehicle that would be safe.

The areas of interest for a high speed, tilt-body train are:

1. Track Panel Shift
2. Wheel Climb
3. Vehicle Overturning
4. Rail Rollover
5. High Speed Stability

The first four are related to curving, the last is a phenomenon of tangent track.

The first, track panel shift, can occur when the total truck lateral force exceeds the restraining capability of the track structure. The track shifts to the outside of the curve, and if the movement is appreciable, causes a derailment. The self-steering axles of the X2000 minimized the amount of lateral force exerted by the axles.

Wheel climb results from a high angle of attack of the leading, outside wheel, and is aggravated by speed and high cant deficiencies. Self-steering axles, as on the X2000, will reduce the angle of attack in a curve.

Vehicle overturning is the result of too high of a speed around a curve and can be controlled by vehicle and suspension design. Under all curving conditions, the center of gravity must not go beyond the field face of the high rail.

The factors that can cause the center of gravity to exceed this criteria are speed, degree of curvature, superelevation and side wind.

Rail rollover will occur if the axle lateral force exceeds the ability of the rail fastening system to prevent the rail from rotating.

Stability is, of course, a major consideration for a vehicle in high speed passenger service. If not controlled lack of stability can lead to either rail rollover or wheel climb.

The initial high cant deficiency tests were done on our Philadelphia to Lancaster main line; this area was generally 140 RE bolted rail on wood ties and rated as FRA Class 4. Amtrak, the FRA and ABB jointly selected three curves for general suitability for high speed and high cant deficiency tests. Although rated as class 4, the track had to be suitable for an increase of one track class since we would be operating around them at substantially higher speeds.

In early December, tests were run on these curves at up to 12 inches of cant deficiency, in some cases this resulted in a curve speed increase of over 50%. Runs were started at the existing level of three inches and gradually increased. After each run, the curves were checked by inspectors on the ground and the on-board, instrumented wheel data was checked to ensure that none of the pre-defined safety criteria were exceeded. We also performed tests with the tilt system disabled, and no increases in any of the rail forces were observed, the ride was "interesting" however.

After the tests were finished on the Philadelphia to Lancaster main line, we moved to our Philadelphia to New York main line. In this area, the track is class 6 and is composed of concrete ties on 140 RE welded rail with a posted speed of 125 mph (under an FRA waiver). As before, tests were conducted to 12 inches of cant deficiency. In this area, we had the advantage of testing high cant deficiency and high speed at the same time.

After the high cant deficiency tests were completed, a series of high speed runs, on tangent track, were made. The tests started at 130 mph and progressively increased until the locomotive ran out of horsepower at 155 mph; no sighs of instability were observed. ABB's model predicted, that with our rail head profile and their wheel tread profile, instability would not occur at speeds below 165 mph.

Amtrak's Midway Interlocking is located in our high speed test zone. During the stability tests in December, and on a media demonstration run in January, we ran through it at speeds in excess of 150 mph.

All tests were completed in mid-January, and the X2000 started scheduled revenue service on February 1st at 125 mph and at nine inches of cant deficiency. On February 10th, the FRA ruled favorably on our request to run at 135 mph.

Amtrak has been moving passengers at 135 mph, and nine inches of cant deficiency six days a week, since February 15th, with absolutely no problems.

TRANSPORTATION TEST CENTER UPDATE RESEARCH AND TESTING CAPABILITIES

By: W. B. Peterson*

The Transportation Test Center (TTC) located near Pueblo, Colorado, is a world-class intermodal research and test center offering a wide range of capabilities for research, development, and evaluation testing of ground transportation systems. This federally-owned, 52-square-mile facility is operated and maintained by the Association of American Railroads, Research and Test Department, under a long term care, custody, and control contract with the Federal Railroad Administration.

The Test Center is self-supporting with revenues from a wide range of customers including the Government, freight and passenger railroads, transit and commuter agencies, car and locomotive builders, track and structure material suppliers, and hazardous materials training students. The TTC is an excellent example of the successful privatization of a federally-owned facility. As a result, the long-term viability of an essential transportation research and testing resource to meet both government and private sector needs is ensured.

This presentation includes a general description of TTC research and testing resources and provides an update of capabilities specific to meeting the needs of the passenger, commuter, and transit industry.

Test Tracks

The TTC site includes 48 miles of railroad track devoted to testing all types of rolling stock, track components, and signal and safety devices. Included among the test tracks are the High Tonnage Loop for track component wear and fatigue research, the Railroad Test Track and Transit Test Track for high-speed vehicle testing, the Precision Test Track for vehicle testing on perturbed track, and the Wheel/Rail Mechanism Loop for steady-state and dynamic curving tests. These tracks are briefly described as follows:

- High Tonnage Loop—2.7 miles
  Track component reliability, wear and fatigue research and testing.
- Railroad Test Track—14.7 miles
  High-speed vehicle stability and endurance tests; evaluation of AC electric-powered locomotives and commuter cars. Overhead catenary system energized at 12.5, 25 or 50kV.
- Transit Test Track—9.1 miles
  Transit car performance and specification compliance testing utilizing third rail, 1,200 volt, DC power and two miles of light catenary.
- Precision Test Track—6.2 miles
  Vehicle dynamic behavior and safety compliance tests.
- Wheel/Rail Mechanisms Track—3.5 miles
  Rail vehicle safety compliance tests.

These and other specialized test tracks are used daily to evaluate vehicle stability, safety, life-cycle reliability, and ride comfort. Past tests have included vehicles as varied as light rail transit cars, an eight-axle nuclear cask car, and high speed passenger locomotives. Utilization of TTC test tracks eliminates the interference, delays and safety problems accompanying testing on an operating railroad or commuter or transit systems, allowing the efficient, economical and safe completion of testing procedures under totally controlled conditions.

*General Manager TTC, Association of American Railroads.

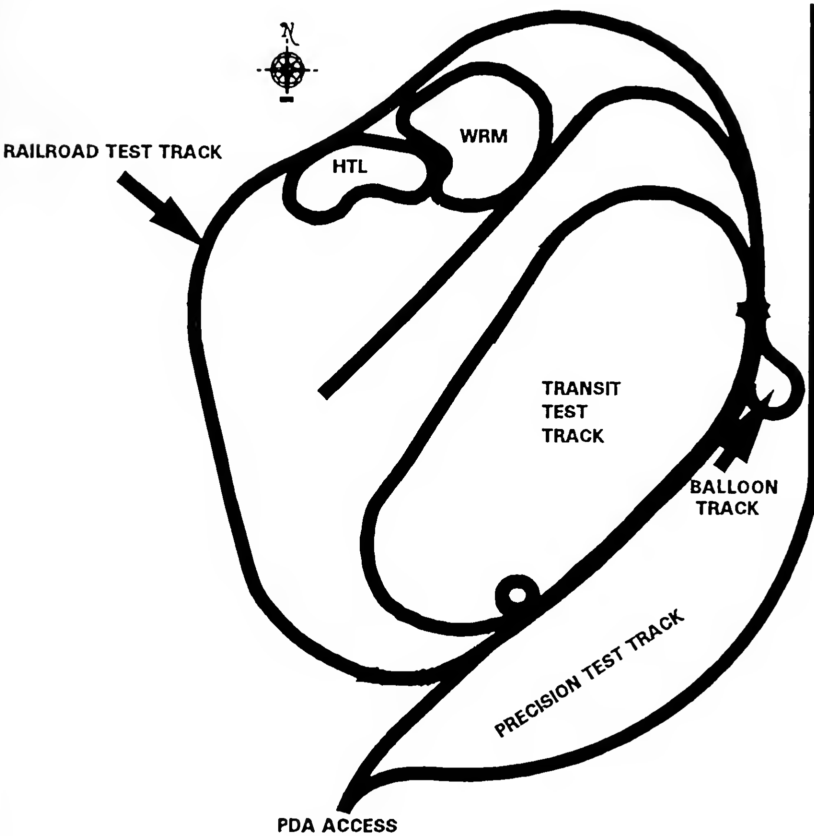


Figure 1. TTC Test Tracks

Test Laboratories

The Test Center also has one-of-a-kind laboratory test facilities used for evaluating vehicle dynamics and structural characteristics as described below:

- Vibration Test Unit

  The VTU subjects full-scale vehicles to controlled, independent or combined vertical and lateral vibrations, thereby subjecting a fully-loaded railcar to simulated on-track conditions. The VTU accommodates four-axle rail vehicles up to 90 feet long, weighing 160 tons, with a 66-inch wheel gage.

- Roll Dynamics Unit

  The RDU evaluates vehicle and locomotive operating characteristics such as vehicle stability, rolling resistance of trucks, and AC or DC locomotive performance. Speeds as high as 140 mph can be applied to test vehicles up to 108 feet long, weighing 200 tons with track gages of up to 66 inches.

- Simuloader

  The SMU performs full-scale vibration and fatigue testing of rail vehicles, buses, or other heavy structures. The SMU inputs motions directly into the vehicle's car body through the car body bolster and is designed to run efficiently for long periods of time making it particularly well suited for fatigue testing. Thus, the fatigue life of a railcar (average of 30 years) can be determined from data obtained with just a few weeks or months of testing. The resulting fatigue analysis serves as an excellent source of design information and safety evaluations.

- Mini-Shaker Unit

  The MSU measures rail vehicle truck suspension system characteristics through the application of fixed or variable frequency vertical, lateral, or roll inputs to one end of the car.

- Traction Motor Test Strand

  The TMTS evaluates the performance of DC traction motor components to study degradation from age and to evaluate the effectiveness of rebuilding procedures.

Rail Vehicle Dynamic Modeling

A number of AAR developed computer simulation programs are used by TTC engineers to predict vehicle and train performance. NUCARS, TOES and TEM are three such programs.

- NUCARS (New and Untried Car Analytic Regime Simulation)

  NUCARS is a generalized rail vehicle dynamics model used extensively both in North America and internationally in vehicle design evaluation, performance prediction, accident investigation, and research. It provides a detailed model of vehicle response allowing the user to easily model any type of rail vehicle from a simple four-axle passenger coach to a complex multi-platform articulated trainset. Vehicle response can be predicted for any sequence of curves, spirals, tangents, and track irregularities. Output variables include wheel/rail forces, suspension strokes, wheel and axle lateral to vertical force ratios, and body displacements.

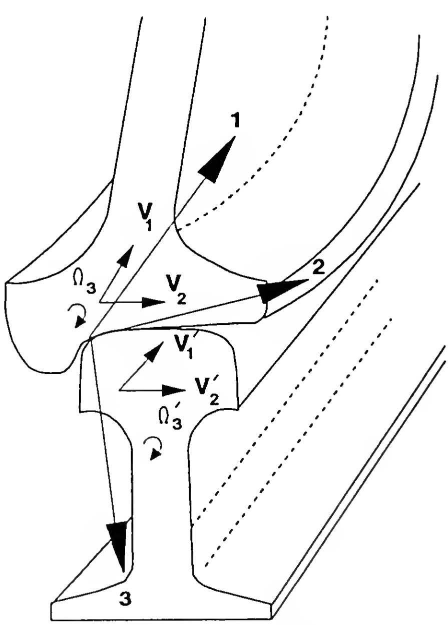


Figure 2. Wheel/Rail Geometry

- TOES (Train Operation and Energy Simulation)
  TOES is a train action model that predicts the response of a train to different operating characteristics such as train make-up and train handling. It simulates air brake systems as well as inter-car coupling characteristics, locomotive response, and train resistance forces. Predicting braking system response, stopping distances, and post-accident reconstruction of train handling are useful applications.
- TEM (Train Energy Model)
  TEM predicts train energy consumption for a given trailing load, route, and speed profile. Applications of the program include scheduling, train operations, and economic studies. As part of program validation tests, locomotive fuel consumption ad exhaust emission rates have been measured.

Applications of these computer modeling capabilities include:

- Accident/Derailment Investigation
  Derailments involving vehicle and train dynamics are analyzed using TOES and NUCARS. Simulation techniques provide an understanding of the wheel/rail forces and displacements generated under variable conditions representing those at the derailment site, such as primary suspension failure, stuck brakes, lubrication and track geometry.
- Suspension Design
  Modeling is an effective aid in the design or redesign of vehicle suspensions to improve lateral stability, ride quality, and curve negotiation. Such analyses are considerably more cost effective than continual fabrication and testing of prototypes.
- Design Evaluation
  The evaluation of existing designs through computer modeling allows the prediction of vehicle response to specific track conditions. Vehicle ride comfort, operating safety and track worthiness can be analyzed and evaluated prior to testing.
- Vehicle/Track Interaction
  Modeling vehicle response to track is important in solving vehicle/track interaction problems including track worthiness of vehicles in spirals and curves, on irregular track, or through turnouts and other special trackwork. Car body motions induced by irregular track not only affect operating safety, but can also degrade passenger comfort particularly with high center-of-gravity passenger cars susceptible to roll inputs.

Vehicle Testing

The Test Center's vehicle testing capabilities cover a full range of characterization, performance, and safety tests for railcars and locomotives in passenger, freight, commuter, and transit service. For example:

- *Characterization tests* determine suspension characteristics and vehicle modal response.
- *Vehicle safety tests* include vehicle track-worthiness testing, derailment studies, and structural certification testing.
- *Vehicle performance studies* measure car rolling and aerodynamic resistance, passenger car ride quality, vehicle load environment, and lading damage.
- *Propulsion system tests* study the function of wheel slip control systems and prototype AC traction systems.
- *Braking system tests* investigate blended braking on passenger equipment, slide control systems, wheel heating, stopping distances, and undesired emergencies in long trains.

The Test Center has provided extensive on- and off-site consultancy services and test support to passenger, commuter, and transit systems. Examples of TTC's experience in performing vehicle testing for both heavy and light rail passenger systems include the following:

- *Vehicle Performance/Suspension Characterization*
  —Ride quality and body vibration
  —Wheel/rail wear and vehicle stability
  —Primary and secondary suspension stiffness and operation safety
  —Train resistance evaluations; i.e., rolling, curving, and aerodynamic resistance
- *Track-Worthiness/Vehicle Safety*
  —Lateral and vertical wheel/rail force analysis of high speed locomotive
  —Performance of different vehicle suspension systems over perturbed track
  —High speed stability performance of modified passenger truck suspensions
- *Structural Integrity*
  —Vehicle body structural strength
  —Vehicle fatigue analysis
  —Shock and vibration levels
- *Ride Quality Evaluation*
  —Transit vehicle ride-quality tests ISO 2631
  —Over-the-road passenger ride quality testing
  —Analyze effect of wheel gages and wheel profiles on rider comfort
- *Energy Consumption*
  —Power consumption and braking efficiency
  —Traction motor power distribution
  —Coast-down rolling resistance
- *Propulsion/Brake System Performance*
  —Stopping distance tests under varying adhesion levels and braking system adjustments
  —On-track propulsion and braking system evaluations
- *Wheel/Rail Interaction*
  —Lubrication practices
  —Wheel and rail profile development
  —Derailment analysis/prevention
  —Curving performance
- *Revenue Service Simulations*
  —Reduced headway studies
  —Schedule performance analysis

Passenger, Commuter, and Transit System Customers

The projects TTC undertakes are not restricted to on-site testing and analysis. TTC excels in off-site work and has conducted major test programs overseas as well as at numerous sites in North America. Passenger, commuter, and transit system customers include:

AMTRAK	National Railroad Passenger Corporation
BART	Bay Area Rapid Transit, San Francisco, California
LACTC	Los Angeles County Transportation Commission
LIRR	Long Island Rail Road
LUL	London Underground Ltd., London, England
MARTA	Metropolitan Atlanta Rapid Transit Authority, Atlanta, Georgia
MBTA	Massachusetts Bay Trans. Authority, Boston, Massachusetts

METRO	Sao Paulo Metro, Sao Paulo, Brazil
NFTA	Niagara Frontier Transit Authority, Buffalo, New York
NJT	New Jersey Transit
SCCTC	Santa Clara Cty. Trans. Commission, Santa Clara, California
SCRTD	Southern California Rapid Transit District
SRTD	Sacramento Regional Transit District
TRI-MET	Tri-County Metro. Transportation District, Portland, Oregon
WMATA	Washington Metropolitan Area Transit Authority

Summation

I appreciate and thank the American Railway Engineering Association and AREA Committee 12—Rail Transit for the opportunity to present this Transportation Test Center update and to describe some of the important work we have successfully carried out for passenger, commuter and transit systems both in North America and internationally. We look forward to expanding these essential services in meeting your research and testing needs.

HOW NORFOLK SOUTHERN AVOIDS SPEED RESTRICTIONS ON CWR DUE TO HEAT

By J. W. Farmer II*

It is my understanding that Norfolk Southern was asked to explain how we avoid speed restrictions on CWR due to heat, after it was observed that our passenger train routes never operate under a blanket slow order, during the hot summer months.

Let me say right up front, Norfolk Southern has developed a program that works for Norfolk Southern, and we are not too proud to improve our program as the need arises. However, we do not mean to suggest that our present program will fit all the needs of any other railroad.

Norfolk Southern has had fair success in preventing buckled track derailments, without the use of blanket slow orders in hot weather. We have had some problems, but we think we have addressed those problems to prevent repeat cases.

Similar to the evolution of the Operating Rule Book, causes of buckled track led to the development of the Norfolk Southern Standard Procedure 390: Maintaining Track Stability. A few copies of the procedure, in booklet form, are available out in the hall, for those interested.

To see why blanket slow orders in hot weather are not necessary, we must examine Standard Procedure 390. In the interest of time, I will briefly discuss the major points of the procedure.

In order to maintain track stability, Norfolk Southern has found 4 factors necessary in the proper handling of continuous welded rail.

1. Lay It Right
2. Maintain It Right
3. Inspect It Right
4. Training

Lay It Right

Initially, rail must be laid right in order to stabilize thermal forces.

We are very proud of the production of our dual rail gangs and the level of quality they are able to sustain. Obtaining proper rail laying temperature is given high priority.

Curve Worn Rail

However, laying one strand of properly adjusted rail is just as important as laying 30 strands, when it comes to controlling thermal expansion. The same rail temperature requirements apply to transposing and replacing curve worn rail by smaller gangs.

Maintain It Right

Even if rail is laid correctly, it must still be maintained properly through many maintenance cycles to insure track stability. In order to maintain it right, the second key point, specific guidelines have been established to cover all maintenance activities. In all cases, CWR must not be disturbed without a proper slow order.

In the area of production work, surfacing gangs must perform all work under a slow order of 25 MPH maximum, with the passage of at least one train to settle the track before the slow order can be raised. A 10 MPH slow order must be used, when rail temperature is expected to be 110° or higher during the work period, and the passage of two tonnage trains is required before the slow order is raised.

*Chief Engineer—Line Maintenance-West, Norfolk Southern

We now have three track stabilizers assigned to various gangs. Where these machines are working, surfacing work may be performed with a slow order of 40 MPH maximum, or 25 MPH maximum if the rail temperature is expected to be 110° or higher during the work period.

The passage of two tonnage trains is still required before the slow order is raised or removed, even though tests have shown that operating a track stabilizer is equivalent to the passage of 7-1/2 unit coal trains, or 100,000 tons.

Timbering and surfacing work, or T&S work, is performed under the same slow order guidelines as surfacing work, with the following additions.

If ties are installed at a rail temperature of 110° or higher, the slow order must remain in effect for at least 2 days of traffic. Newly installed ties are to be spiked, tamped and rail anchors applied at time of installation. At the end of the work week, all disturbed track must be fully tamped, and ballast section restored.

Rail temperature is critical with respect to lateral stability and for this reason it is the factor which activates many of our guidelines. All production gangs are required to take rail temperatures a minumum of 3 times daily. These temperatures are reported along with the production reports to our Atlanta office.

In the early 70's we experienced several buckled track derailments on curve locations that had been surfaced the previous winter. Because the track had been worked below the rail laying temperature, the disturbed track moved inward on curves during the work and settlement period. No record was made, and so no adjustment resulted. For this reason, each spring and summer we had track alignment problems.

To prevent these problems, instructions were written to measure movement of curves disturbed during cold weather.

Where track will be surfaced at a rail temperature of 50° or below, reference stakes are to be set on curves and measured ahead of the work. One week behind the production gang, measurements are taken to record any movement of the curve. This information is sent to the division engineer, who is responsible for adjusting rail on all curves which average one inch or more inward movement.

Measurement of curve movement has been a very good practice with positive results. Ideally, we prefer not to work track at below the rail laying temperature; however, realistically, this is not possible and the practice of measuring our curves for inward movement has prevented many problems.

Spot maintenance work is performed under the same slow order guidelines as production work. Additional restrictions are dependent upon the nature of the work.

If more than 4 continuous ties are hand tamped, a 25 MPH slow order must be in effect until track is power tamped. When smoothing or restoring prescribed elevation in curves, each tie must be fully tamped under each rail to eliminate voids between the tie and ballast section.

Welded rail should not be smoothed when rail temperature exceeds 110° unless such smoothing is necessary to afford safe passage of trains.

Spot replacement ties or switch ties must be installed under the same slow order guidelines as for T&S work. Also, if more than two ties are installed per 39 foot rail length, the ties must be power tampered before the slow order is removed.

Cribbing and spot undercutting are regulated by the same slow order guidelines as for surfacing. Undercutting out of face requires a 10 MPH slow order for a minimum of 24 hours of traffic. Thereafter, speed may be increased to a maximum of 25 MPH for 2 to 4 days, depending on traffic volume. Any tangent track that cannot be restored to proper alignment during the heat of the day must have the rail cut and adjusted, before the slow order is raised.

Measurements are taken for possible inward curve movement in conjunction with undercutting if the rail temperature is 70° or less during the undercutting and surfacing work.

The bridge work procedure has undergone considerable revision in recent years. When renewing bridge ties on open deck bridges, a 10 MPH slow order must be used until all rail anchors and fasteners are re-installed.

If rail displays tightness or if rail temperature exceeds 95°, no more than 10 ties are to be unspiked at any one time, unless:

1. Rail is known to be adjusted to a higher temperature.
2. Rail is cut off the bridge, and allowed to move freely.
3. Rail is known to be moving freely in expansion joints.

Double jaw gauge rods must be used to secure rails on thru girders and thru trusses, if more than 10 ties need to be unspiked, above 95°. A 25 MPH slow order must be used until all guard timbers are installed and ties are seated by the passage of two tonnage trains.

The final maintenance process to be discussed is field welding which has a potential to add rail to the track, thereby lowering the effective rail laying temperature at a specific location. Norfolk Southern presently uses two field welding processes.

We have two flash butt welding trucks, and are presently building a third truck, for rail laying and out of face welding of jointed track. Rail heaters are to be used to insure proper rail adjustment with these high production units.

The other process is thermite welding, which can make one to several welds at a given location. When field welding is at a rail temperature of 75° or less, rail movement and the amount of rail added or removed is recorded. Rail must be adjusted as necessary prior to hot weather.

Inspect It Right

Track inspection is our first line of defense for detecting any flaws in the track. We are fortunate to have officers who are very knowledgeable track maintenance personnel, assigned to all track inspection positions.

There are two types of inspection. First, all scheduled track inspection must be maintained. Special attention must be given to track on curves, in dips, at the end of bridges, heavy grades, recently disturbed track, track worked during the past winter, or locations of multiple thermite welds made during the past winter.

Second, additional inspections are to be made weekends and otherwise, during extreme high or sudden changes in temperature, where rail or recently disturbed track is subject to getting out of line.

We must be fully aware of the situations that disturb the track and cause a loss of resistance to lateral forces. When this work is done with rising or high rail temperatures, extreme caution must be taken to prevent track buckling situations.

Once again, we are blessed with dedicated officers, who spend a great deal of extra time making additional inspections to protect our railroad in all types of weather.

Tight rail is rail not properly adjusted and is considered a track defect on Norfolk Southern. Whether detected by inspection or encountered during scheduled work, tight rail must be adjusted or slow ordered until adjustment is complete. When track is known to be tight or has moved out of line at the end of a bridge without expansion joints, the rail must be cut and adjusted rather than lining the track.

Each spring prior to warm weather,division engineers must review all records of track work since the previous fall for locations of rail requiring adjustment. Rail laid below desired temperature, curves that moved inward an average of one inch or more behind surfacing, and field welds where substantial rail was added must all be adjusted by cutting or lining out prior to hot weather. If these adjustments are not executed, track is to be slow ordered in warm weather until lining is complete. In most situations rail adjustments are made prior to the need to slow order.

Training

Last but certainly not least, Norfolk Southern believes that training is the backbone of this successful program. Standard Procedure 390: Maintaining Track Stability was developed in the mid 70's to provide precise handling of how to avoid track buckling problems of the past.

Norfolk Southern began holding annual spring hot weather meetings in 1974 for all MW&S officers, with the technical support of our own Research and Test Department.

As long as I work, I'll always remember one of this association's own distinguished members, Charlie Scott, opening the first meeting by saying:

"Gentlemen, It's springtime. The birds are singing, the bees are buzzing, and the track is buckling."

That corny message alerted everyone to be on guard for the coming of warm weather.

Every year during the meeting, Standard Procedure 390 is covered word for word.

Graphic illustration of actual buckled track derailments serves to reinforce the need for strict compliance to procedure. What more do you need, than to see an Amtrak coach off the track and in the ditch?

In addition to Procedure 390, certain track stability factors are also covered for better knowledge and understanding. It is made quite clear that track disturbed by surfacing can lose as much as 80% of its original lateral restraint and can only be recovered by tonnage at reduced speed or by use of a ballast stabilizer.

Certain track conditions are discussed to insure strict adherence to the standard. Maintaining proper ballast section is emphasized to afford necessary lateral stability.

To control the compressive forces of rail expansion, no part of the track is more important than the rail anchor. Anchors must be applied to standard. All missing or defective anchors are replaced in each timbering cycle.

The rail anchors serve no purpose unless they are boxed against the crossties. Therefore, each timbering and surfacing gang is equipped with machines to squeeze all anchors tight against the ties.

In recent years, hot weather training has evolved to include all scheduled employees who require FRA certification. Field classes are held each year immediately after the officer hot weather meetings.

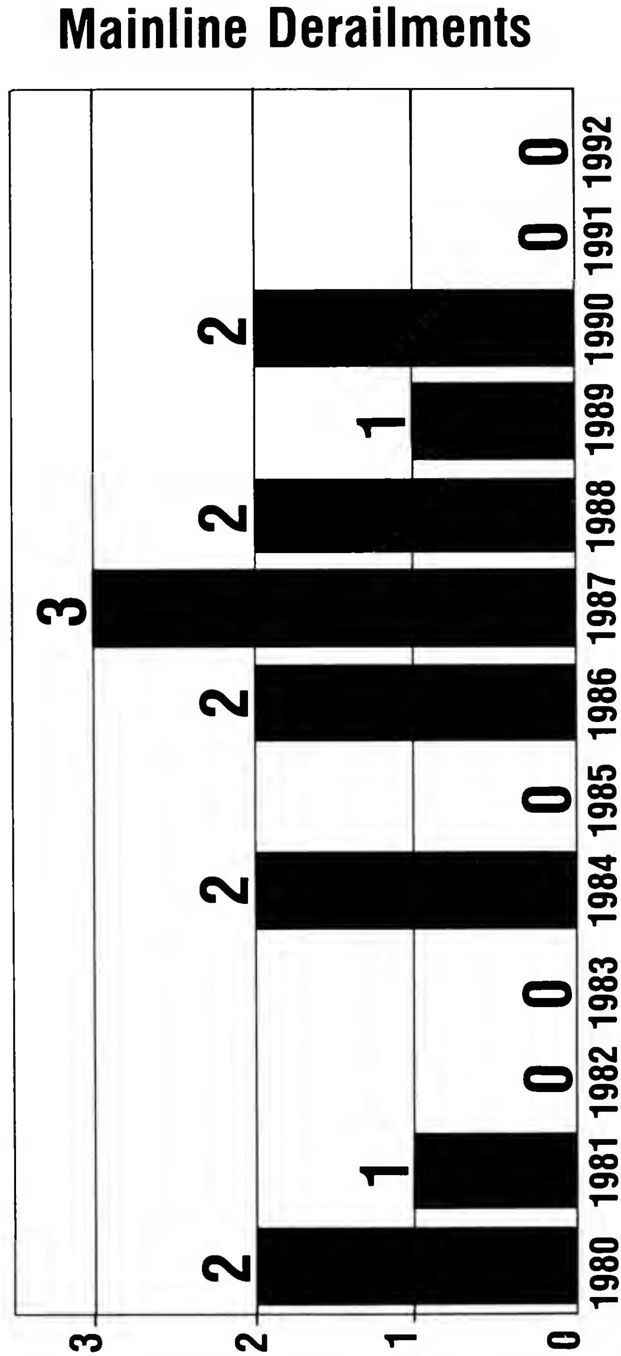
For further training, all foremen, assistant foremen and new supervisors attend a two week training school at the Norfolk Southern Training Center at McDonough, Georgia. This training includes an indepth study of track stability, and all employees must take and pass an exam on FRA Track Safety Standards.

In summary, Norfolk Southern slow orders are used during track work and the settlement period as an aid to strengthen the track structure and improve track stability. This policy enables the track to maintain maximum speed regardless of the weather. We may not run the fastest trains, but we try to run them more often.

I'll conclude with the statement that we use to close all hot weather meetings.

Disturbed track in hot weather,
plus failure to follow instructions,
equals buckled track.

BUCKLED TRACK DERAILMENTS
(Through 1992)



STATUS OF THE FRA BRIDGE SURVEY

By: G. A. Davids*

There is always some trepidation in all quarters when we start a project like this in an area that is essentially new ground for the FRA. I know many of you wondered what FRA would do with our track inspectors out looking at bridges, and how we would use the information from the survey.

Well, we are far enough along with this now that a pretty clear picture is developing. First of all, and this is why I am so pleased to be here, this entire project has worked better than any of us could have imagined at the start. The FRA track inspectors assigned to the bridge program have received some excellent training and experience from the training courses we ran for them and from working with the railroad bridge engineers and inspectors during the past year. We have seen and documented some very comprehensive bridge inspection and management programs on most railroads.

FRA's Concern with Railroad Bridges

Why is FRA showing so much concern for railroad bridges right now? First of all, I can tell you that there is no external pressure on FRA to be doing this. We are not responding to any particular Congressional mandate, nor to any other initiative from outside the FRA. We have, however, seen an increasing number of questions and complaints, justified or not, about the safety of railroad bridges.

The issue of "Decaying Infrastructure" is receiving considerable attention throughout the Nation. This issue can easily lead to sweeping generalizations. Public perception can quickly diverge from reality, and there is a danger that public policy might be based upon false perceptions unless reality can be documented. We saw an urgent need to set a policy on bridge safety, based on reality rather than perception.

People are interested in railroads. They are big, they are everywhere, and railroads are just plain interesting to many people. Bridges are among the most visible features of a railroad. Bridges are big, they are visible, and some of them are not especially beautiful.

To the layman, rusty steel or spalling concrete means that a bridge is in trouble. Some cannot believe that a wooden bridge can possibly carry something as heavy as a train. We get letters, and we are obligated to investigate and answer them.

We have to be able to give good answers to those complaints and questions. If we cannot give good, factual answers, there is a real possibility that public opinion and policy might be developed from perception instead of reality.

The Need for an FRA Bridge Policy

An unfavorable incident involving a railroad bridge would undoubtedly cause demands for some type of action by the government. The best way to prevent hasty, ill-conceived action is to prevent the incident in the first place, so the primary objective of an FRA bridge policy should then be the prevention of bridge failures.

The secondary objective should be informative. Good information should be made available to all concerned. The public should know that railroads are managing bridges in a responsible manner, if that is truly the case. The railroads should know if FRA sees a problem with their bridge management.

A future FRA bridge policy might or might not call for some form of regulations. If it should, then the policy statement would be found in the regulations and supplemental information published in the Federal Register.

But here is the interesting part. FRA can gather information formally or informally that supports a decision not to issue regulations. We can publish the reasoning behind that decision in a statement of policy.

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*Bridge Engineer, Federal Railroad Administration

In any case, an FRA policy on railroad bridges must be based on well documented facts and sound engineering. It should incorporate the best information available from all competent sources and interests. It should be understandable and useful, and it should promote railroad safety. It should not disrupt the many good railroad bridge management programs now in place, and it should not disrupt the economic advantages of railroad transportation.

How FRA is Developing a Bridge Policy

We are developing this policy by gathering facts about railroad bridges and bridge management. We started the survey in April 1992 and we plan to finish around May of this year. After the survey is done and the facts are in, we will draft a policy that will be supported by the facts, and which will support FRA's mission to protect the public safety. The policy will then be published.

Purpose of the Survey

The primary purpose of the survey is to determine and document whether or not there is any significant railroad bridge safety problem in the United States. We also want to find out just how railroads are managing the structural integrity of their bridges and how those programs affect the safety of the bridges. The results of the survey will provide the basis for an FRA bridge policy. Because the policy should be designed to accommodate good practice, we are gathering information on bridge management practices so that we can write a policy that will accomplish that goal.

Who Conducts the Survey

The survey is being conducted by the track specialists and about thirty track inspectors in the eight FRA regions.

The inspectors' background, of course, is primarily in track rather than bridge work. We did not turn them into "two-week wonder" bridge inspectors. We did train them to recognize their limitations.

They have been through two-week training courses on bridge theory and inspection during February and December, 1992. The training was conducted by experienced railroad bridge engineers with the invaluable assistance of CSX, Norfolk Southern, and Conrail. We spent about half of the time in the field on bridges in the vicinity of Atlanta, Georgia and Baltimore, Maryland.

The inspectors understand bridge terminology, behavior of bridges under loads, and the particular difference between ugly and critical conditions. They are all good track inspectors, and they have prove particularly adept at locating bridge problems that show up in track geometry. This is a valuable skill.

All of the railroad people with whom they have worked have been extremely helpful and cooperative. The FRA people have gained some highly valuable practical experience by working closely with the railroads' inspectors and engineers. This is a vital point. We know that this project could not have succeeded without the wholehearted cooperation of all the railroad employees, engineers and officials with whom we have worked. This positive, open attitude will permit the quick resolution of any bridge problems.

Why FRA Has No Specialized Bridge Inspectors For This Work

FRA has not hired any additional personnel to conduct the bridge survey. When I became the Bridge Engineer, I came over from the Office of Research and Development, and FRA did hire a replacement for me, but that is the only staff increase. It would be neither fact nor good public policy to hire more people for a bridge program unless there is a clear, long term requirement for their service.

There are no FRA employees whose jobs depend on the growth of a bridge safety program, so no one has a vested interest that would influence the results of the survey, or FRA bridge policy. And I have plenty of work to do, regardless of the outcome.

There is an advantage to having a number of track inspectors around the country who, on fairly short notice, can handle bridge questions as they arise. We save considerable travel time and money, compared to using a relatively small group of bridge people who would be traveling away from headquarters most of the time.

With the cooperation of the railroad bridge engineers, and some guidance from our office, they can move quickly and get all the information we need in most circumstances. I think this will work well as long as we keep the program on an informational rather than a regulatory basis.

There is another advantage to giving our track inspectors some indoctrination in bridge work. We have had a few cases already when a track inspector with bridge training was making a routine track inspection and, thanks to his understanding of track and bridges, was able to detect a small track deviation which led to detection of a fairly serious bridge problem. Another pair of good eyes never hurts.

FRA Inspectors Are Not Actually Inspecting Bridges in the Survey.

The FRA inspectors are observing the inspections being made by the railroad inspectors. They are not actually conducting bridge inspections. If we tried, it would take years, it would amount to duplication of effort, and the benefit would be questionable. The responsibility for the safety of a bridge rests entirely with the bridge owner, and that responsibility should not be shared with FRA or any one else.

Extent of the Survey

The survey includes:

- All 19 Class I railroads, or railroads that were Class I when we started.
- 7 passenger and commuter railroads, including Amtrak.
- 11 selected regional railroads.
- 41 selected Class III railroads.

The selection process was fairly random, designed to include a range of railroad sizes and operating conditions, spread over the 8 FRA regions.

Progress of the Survey

Of the 78 railroads in the survey, we have interim reports on 54, and final reports on 34, as of March 9. We had hoped for completion in March, but will probably go until May because of weather conditions. There is no external pressure for a deadline, so we can take the time to get this right the first time.

How the survey is organized.

Each selected railroad is assigned to a principal FRA region for contact and report preparation. Railroads that extend through several regions see inspectors from those regions for the field work. On large railroads, the track specialist from the principal FRA region has met with the bridge engineer designated by the railroad, received an overview of the bridge inspection and management policy, and incorporated that into an interim report.

The interim report is used as a coordinating tool and a primary source of information for the survey. It is a characterization of the railroad's statement of bridge management policies and practices. On smaller railroads that are handled by only one inspector, an interim report is usually not necessary. Their programs are necessarily simpler, and usually, the entire survey can be conducted in just a few days.

The field work (observation of inspections) is done primarily to verify the statements in the interim report. The assigned FRA track inspector travels with the railroad inspectors and observes their work. We have tried not to disrupt the railroads' normal bridge inspection schedules, which is one reason we allowed one year for this survey.

Our inspectors complete a regular track inspection report listing the bridges, because FRA is a federal agency and they must have some paper to show for their efforts. They also make a bridge report, which is actually a cover page showing basic data on each bridge, such as type of structure, span length, and location. There is not much requirement for detail about conditions on that form unless something is especially notable.

All of the information gathered by the inspectors and specialists is to be shared with the railroad before it is sent to Washington. We want to know that the information is factually correct, and we certainly want the railroad to know as soon as possible about any problems our people might have found regarding either their structures or their programs. Of course, out on a bridge this information would pass immediately between the FRA and railroad inspectors, but even in that case the railroad people will see a copy of the FRA report as soon as it is prepared.

What the Survey Has Shown So Far

First, and most importantly, we have definitely found that a very large majority of railroad bridges in the Nation are inspected at least annually by competent inspectors, and that most large railroads have current records of the actual condition of their bridges. We have not found any immediate hazards caused by negligent or irresponsible behavior of any railroads.

Many of the FRA inspectors have made it a point to comment on the professionalism demonstrated by the railroad inspectors with whom they have worked. Our people do not make these comments lightly, so this truly amounts to high praise.

We have found that many railroads are now computerizing bridge inspection and management information. This is working well in most cases, providing good management information while reducing the time required for paperwork.

I can tell you that there is no plan to require hard copies of bridge inspection reports, in the manner of the Track Safety Standards, signed by the inspector. Do not hold back on the adoption of electronic data processing of bridge information out of fear that FRA might render your system useless.

Some inspection records do not show the date that an inspection was actually made, and who made it. Without this information, it is difficult to assign responsibility for inspections, and to determine if inspections are being conducted at the prescribed frequency.

Some railroads have assumed ownership of bridges without receiving drawings and records. This, of course, complicates the correction of any future problems on these bridges and the implementation of bridge management programs. I would strongly advise anyone planning to acquire railroad property that they be certain to get originals or good copies of the bridge drawings, design calculations, and maintenance records. If copied, they should make sure they know who has the originals.

We found several smaller railroads that had recently started their bridge management programs. We found some that have none. Many of the smaller railroads do have very adequate bridge management programs, which are well suited to their size and to the nature of their bridges. Many of them use a combination of experienced consultants and their own personnel for inspection and maintenance. This is not a difficult undertaking, and we would recommend to those railroads with no program that they would do well to contact their neighbors and organize a practical bridge management program.

Everyone should understand that railroad bridges are not highway bridges. Railroad loads, designs, details and response to loads are considerably different. Highway bridge inspection and analysis methods cannot be successfully applied across the board to railroad bridges. This is a major factor to consider in the selection of consultants.

We found wide varience in policies for underwater inspection of substructures. On some railroads the annual inspection includes rodding, probing or sounding the stream bed around piers and abutments. Specific procedures and schedules have been developed for larger or more complex structures. On some other railroads, there was less evidence of records of conditions under water. We believe that an underwater inspection program is essential to bridge safety.

In at least one circumstance, bridge inspectors and even the bridge engineer were not authorized to take a bridge out of service, or even place operating restrictions without the approval of the transportation department. We feel strongly that the personnel responsible for the safety of bridges must have authority commensurate with their responsibility.

Not all railroad bridges can be expected to provide trouble-free service for the remainder of their possible lives. We realize that most railroads have some "problem bridges." This is not alarming, because we now find that safety can be maintained by appropriate levels of inspection and monitoring by the bridge owner.

Some of these problems will become increasingly expensive to correct over time, but as long as this can be accommodated with safety, we view this as an economic problem. However, if these ever begin to affect the excellent safety record of railroad bridges, FRA will be obligated to take other action.

Most importantly, we have seen no indication that there is a significant safety problem with the structural integrity of railroad bridges. Safety is being maintained by competent engineers with accurate information on the condition of their bridges.

The Future

Unless there is a major change in safety policy at FRA, we plan to publish the general findings of the survey in 1993, together with an official statement of FRA policy on the subject of railroad bridge integrity. The policy will include justification for any action that FRA does or does not take on railroad bridges. If regulations are not required, the reason will be stated.

The policy might include informal guidelines that would establish a framework by which FRA or any railroad could evaluate railroad bridge management procedures and practices.

We will probably continue to maintain contact with railroad bridge engineers so that our information will be reasonably current. There are also many smaller railroads that were not included in this survey. We plan to contact the remaining small railroads over the next several years.

Conclusion

I think that most of this has been good news. We are very much aware of the excellent safety record surrounding railroad bridges for the last half century or more, and we certainly do not want to change this. So far, we have not found any bridge problems that we cannot handle on a case-by-case basis with the railroads involved.

Finally, I would like to say that we have received not just cooperation but extremely active assistance from the railroad bridge people in this endeavor. Your positive attitude toward our work has made this a very pleasant and worthwhile experience for all of us at FRA who have been working on this project. On behalf of all of them, I would like to express our personal appreciation, and to thank you all very much.

INFRASTRUCTURE AND TELECOMMUNICATION CHANGES IN THE MEXICAN RAILWAYS

By: G. Rivera D.* and E. Barousse**

GROWTH THROUGH PRIVATIZATION OF THE NATIONAL RAILWAYS OF MEXICO

The History of the Mexican Railways

The turn of the century saw an increased role of the private sector in running the railroads of Mexico. The Mexican government began to permit private companies to build and operate the rail lines which connected Mexico's most important centers of production with seaports on the country's Gulf and Pacific coasts, and its northern (U.S.) border. In this manner, over 20 railway companies came about and, relying on their own organizational system, competed among themselves for the overland transportation market.

The National Railways of Mexico began in the 1909 merger of the railway companies which operated in the Central and Eastern areas of the nation at that time. The company had a total of 10,000 kilometers of line, and adopted a centralized system of organization.

Later, more companies were integrated into the National Railways of Mexico and, up until 1986, there existed only three other railway companies. They were: The Sonora-Baja California Railways, Inc.; The Pacific Railways, Inc.; and The Chihuahua-Pacific Railways, Inc., whose central offices were located in the City of Chihuahua. In November of 1986 they merged with the National Railways of Mexico. National Railways of Mexico presently has 20,210 kilometers of main rail lines and 4,500 kilometers of secondary lines.

Changes in the Organizational Structure of the Railways.

Before 1986, The National Railways of Mexico had a centralized system of management consisting of departments divided according to areas of responsibility. The administrative, logistical and support systems were highly centralized. At the same time, the technical operations of the railway system were decentralized and divided by geographical location. This organizational structure proved to be inefficient.

In 1986, the structure was changed, and five regional divisions were created, each covering approximately 4,500 kilometers of main rail line. The regions are: the Southeastern Region, with headquarters in Veracruz; the Central Region, with offices in Queretaro; the Northeast, with headquarters in Monterrey; the Pacific Region, with offices in Guadalajara, and the Northern Region, based in Chihuahua City.

The goal of this decentralization was to make the administrative system more responsive and to increase productivity by granting greater decision-making power at the operational level. Resources were distributed so that decisions could be made on site where problems presented themselves. This eliminated the need to wait for the central authorities to intervene, as had previously been the case.

Effective Decentralization.

Although the groundwork for a new organizational structure was laid in 1986, the changes were not completely carried out. The central offices still reserved for themselves a significant part of the responsibilities which were to be decentralized, including budgeting and spending. Human and material resources were also administered from the central offices.

The present administration decided in April 1992 to implement substantial changes in order to reinforce the regional divisions powers which had been introduced in the initial changes in 1986. With this new reorganization, the Railways, which had been lagging behind the rest of the country in modernization, now joins the movement to make the nation's economy more efficient.

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Under the new organizational structure, the regional divisions now carry out the functions of regional planning, coordination and integration, executive functions, supervision, regional administration and informational duties.

The central offices are now assigned the following duties: norms and procedures, planning at the national level, inter-regional coordination, the execution of supra-regional projects and their control and assessment.

Fundamental Policies of the Present Administration

Previously, the National Railways were considered to be, above all, a social service. Therefore the goverment contributed economic resources to the Railways to make up for income which the railways did not receive from involvement in badly needed, but non-profitable social service.

At this time, the main objective of the National Railways of Mexico is to become financially self-sufficient, thus making it a profitable enterprise capable of competing with other means of overland transportation.

In order to achieve this goal, the National Railways has implemented various measures and strategies to modernize and become more efficient. Among these measures are:

A. Redimensioning of the labor force: Until June of 1992, the Railway employees numbered approximately 80,000. The labor force was reduced by 25% to approximately 20,000 workers by implementing a voluntary retirement program with incentives more attactive than those stipulated by the federal labor laws and the labor contracts.

B. Reduction of the length of the total lines: Out of the Railroad's total of 20,210 kilometers of main lines, 2,400 kilometers of those turn no profit, and therefore should no longer be operated. The rail network will thus be reduced to 17,790 kilometers in length.

C. The upgrading of the basic network: 11,000 kilometers of track need to be upgraded to meet international standards of quality, so that they will be at least equal in quality to those of the United States and Canada.

E. Review of services: An economic analysis of all the runs determined that some passenger service was unprofitable, and therefore 40 of these runs were cancelled, leaving services only on the main rail routes and on the lines in regions where no other means of transportation exists.

   The type of freight which the railways handle has also changed, with a decrease in minerals and ores, and an increase in manufactured products of greater value, such as automobile parts. Long-distance unit trains have thus increased in number, and transportation using double-stack trains has a promising future in the railroad business in Mexico.

F. Private-sector participation: The National Railways of Mexico is following the national trend by adopting a policy of encouraging private sector participation in various projects. These projects include locomotive and wagon repair, the construction of new rail links and new lines, and the operation and construction of railway freight terminals known as 'Ferropuertos.'

G. Improvements to railway installations in seaports and on the borders: The aim of making terminal service more responsive means renovation must be carried out on the rail lines in the seaports and on the borders, particularly at the junctions of trunk lines which carry high traffic loads.

H. Revision of labor contracts: The company has negotiated changes in the labor contracts with the rail union in order to eliminate certain clauses which limit the modernization of service and the participation of the private sector in some of the most important labor functions.

1. The construction of new lines: Although the present priority is the maintenance of the National Railway's existing lines, new lines that have already been started must be finished, particularly the lines which form part of the Guadalajara-Nuevo Laredo main route. The stretch of line between Guadalajara and Aguascalientes is now almost 40% complete.

All of the above actions comprise the new general policy of modernization and efficiency in rail service, creating a new orientation towards the participation of the private sector, in order to facilitate the goal of competing with other means of overland transportation in Mexico.

Advantages of Railroad Transportation in Mexico

Many different modes of transportation complementing each other make up a nation's infrastructure. The railroad is specialized to move large volumes of freight over long distances.

As a result of the latest energy saving policies and pollution-control regulations in Mexico, rail transportation has gained new favor because it uses less fuel per ton of weight carried and pollutes less than any other mass transit system.

Moreover, the weight limitations imposed on highway traffic, combined with the increased capacity of rail lines and bridges to handle 120-ton cars on the main and special trunk lines, are causing freight traffic which had been taken by highway transportation to return to the railways.

Rail transportation is vital to the development of any country, and as the national and international economy grows, rail transportation must become more efficient in order to offer an adequate response to the demands which arise from new stages of development.

The National Railways of Mexico is committed to achieving its goal of structural change. If, in the future, they are able to offer a more efficient service, the Railways will undoubtedly take its rightful place as the backbone of Mexico's overland transportation.

SPECIFIC ASPECTS OF STRUCTURAL CHANGE IN INFRASTRUCTURE.

Organization of the Infrastructure

In 1989, the General Direction of Railways, which had belonged to the Ministry of Communications and Transportation (Secretaria de Comunicaciones y Transporte) was incorporated into National Railways of Mexico as the General Sub-Direction of Construction, and charged with the responsibility of constructing new railway lines and links in order to meet needs for expansion of National Railways of Mexico.

Within the National Railways, there already existed the General Sub-Direction of Track and Telecommunications which basically carried out tasks associated with track and structures maintenance, telecommunications and signals. With the aim of integrating into one single office all those functions regarding fixed facilities, in April of this year the two previously-mentioned sub-directions were merged, giving rise to the present General Sub-Direction of Infrastructure and Telecommunications.

The new General Sub-Direction is basically made up of four operational areas:

The construction area, responsible for the construction of new lines and projects of national scope.

The track and maintenance area, which administers standards and norms for the maintenance and operation of lines and facilities.

The projects area, which takes responsibility for new designs and improvements to existing installations, as well as standards and quality control of work being carried out.

The telecommunications area, which is responsible for the maintenance of signals and telecommunications as well as the installation of new signals and telecommunications.

In each of the regions, the vice-manager of track and telecommunications coordinates and supports the projects carried out by the division's engineers, who are in turn responsible for complying with the programs issued by the regional administration and approved by the central offices. In this way, the basic working groups in the National Railway's 25 divisions which are involved in infrastructure organization

are being reinforced by technical personnel who previously belonged to the central office staff. This is done so that they can fulfill their responsibilities of administration, supervision and execution of all the maintenance and rehabilitation projects in their respective jurisdictions.

Under the new organizational system, the central offices now assume responsibility for norms and standards and with carrying out projects of national scope.

The decentralization policy forsees the acquisition of materials in each of the five regions of the National Railways for use in maintenance and rehabilitation projects, with the exception of imported materials or those available from only one supplier. These will be acquired centrally, as will machinery and equipment.

Reduction in Personnel.

The on-site staff in the area of infrastructure was, until May of 1992, made up of 18,682 employees. This number created a heavy financial burden on the National Railways.

In order to reduce payroll costs and make railway service more efficient, a consistent policy was established to simultaneously modernize maintenance systems and reduce the work force.

Strategies for Maintenance and Renovation.

Of the 20,210 kilometers of trunk lines which belong to the National Railways, 8,100 kilometers of track are laid with welded continuous rails. These rails, placed over concrete ties, vary in size from 100 to 136 pounds/yard. Traditionally, maintenance and renovation on these lines were accomplished through labor gangs or section gangs; however, due to the weight and rigidity of the track components mentioned above, this method was no longer effective. Thus, a radical change in methods of maintenance and renovation became necessary. This was achieved through the integral mechanization of the process: carrying out processes of alignment, leveling and ballasting with machinery. With this end in mind, use established mechanized maintenance and renovation crews with a jurisdictional range of approximately 120 kilometers. Each crew is provided with a hi-rail to cover their section of track and make isolated repairs using the modem and highly efficient equipment which each hi-rail is equipped with. The mechanization program is initially planned to cover the most important trunk lines, such as Mexico-Laredo, Mexico-Guadalajara-Manzanillo and Mexico-Coatzacoalcos. In the first stage of the program, 34 hi-rails are being acquired under the 1992 investment program; in the second stage, which will cover up to 8,100 kilometers of main line, another 34 vehicles will be acquired in 1993 and 1994.

The rest of the rail network, some 12,110 kilometers of standard line with bolted joints and spike fastenings, will continue to be maintained by rail gangs grouped appropriately and supplied with motor cars, hand tools and the necessary machinery to accomplish maintenance and renovation work.

Voluntary Retirement of Employees.

The National Railways of Mexico's new organizational policy includes a program of voluntary retirement which offers greater benefits than those required by law for employees who elect this option. The program's goals have been met to a considerable extent, given that a recent study determined that the practical limit on personnel reductions through this method was supposed to be some 7,900 employees, while the present program has resulted in the retirement of 6,501 field employees. The program of re-grouping the remaining 12,181 line and structural employees for purposes of maintenance and renovation work is nearing completion.

This personnel reduction has allowed the National Railways to apply the savings in payroll to fund longer-range and more efficient programs of maintenance and rehabilitation. These developments fall in line with the goal of achieving a more modern infrastrucutre in a shorter period of time.

Private-sector Participation in the Renovation and Rehabilitation of the Lines.

Until now, all work done on the maintenance and renovation of lines was performed by the railway's own crew. The company acquired the necessary machinery within the rubric of its annual investment budgets, and the railway's own employees would constitute the work crews needed in alignment and leveling and for the reinforcement of earthworks and the cleaning of ballast.

The opportunity for private sector participation is one of the main policies of the present administration. This new policy is in response to the present demand for more efficient rail service, and approval has been given for private sector businesses to participate in the performance of maintenance and renovation, including leveling, alignment and ballasting on the basic rail network. This step allows National Railways of Mexico's existing stock of machinery to be used in servicing the rest of the network. Private sector enterprises are able to provide a more prompt and rapid maintenance and renovation service on long stretches of line due to their competitive nature, greater access to economic and technical resources, and their ability to adjust their work schedules to traffic conditions. They are even able to schedule night shifts or mixed-shifts when the schedules of train traffic in the area require this, and at the same time, achieve high-precision work by employing modern equipment.

The first stage in the program to contract private-sector companies in maintenance and renovation was to choose 11,000 kilometers of priority line within the basic network for work with machinery groups. Each group consists of one multitamping aligning levelling machine, one ballast regulator, one ballast undercutter cleaner machine on some stretches, and one dynamic stabilizer for those lines with the highest traffic density. The stretches designated for the first stage of mechanized maintenance and renovation will be routinely serviced and guarded by mechanized crews travelling in high-rails.

The budget approved for 1992 already includes the first stage of maintenance and renovation with private-sector participation, thanks to new agreements made with the railway worker's union of The Mexican Republic. This represents an important step in the program of structural change which is being implemented in the National Railways of Mexico, allowing progress to be made in these areas using both the high-technology equipment on the main trunk lines and the 34 mechanized levelling equipment groups which the company has already acquired.

Rehabilitation on welded rail over concrete ties will also be executed by private companies using their own equipment and personnel. This work, however, will be under the supervision of and protected by the division engineers of the regional offices. This allows the railway's own crews to dedicate their attention primarily to maintenance.

In the numerous ways shown here, the National Railways of Mexico is successfully implementing a program of profound modernization in accordance with the guidelines of the sector of communications and transportation and our general director. These organizational changes in railway transportation will undoubtedly permit the National Railways of Mexico to become a more profitable and efficient overland transportation business, and answer Mexico's need to develop and to meet the challenge of increased commercial opportunity with the railroads of North America.

THE NEW CULTURAL DIVERSITY: RAILROADS AND POLITICS

By: A. C. Witzig*

Abstract and Introduction

While American railroads have operated within the overall political environment since their inception in the early 19th Century, it is a relatively recent phenomenon that governments are actually stepping into ownership and full operational roles in what were formerly completely private rail routes. This is especially true in metropolitan areas developing new commuter rail operations, in which a rail solution is being invoked to solve a non-rail problem. As could be reasonably predicted, both public officials and railroad operators have been surprised and perplexed by each others' expectations, cultures and operating assumptions. Since the evolving partnerships offer significant benefits to both parties, it is important for each partner to understand the context from which the other comes, be willing to keep communications open and show flexibility in accomplishing mutual goals.

The actual collaboration on a day-to-day basis between government and private business has been and will continue to be an ongoing challenge for both railroaders and public officials. My purpose today is to try to shed some light on the collaborative process and to encourage further progress along this new path.

Traditional Roles

From the beginning, railroads in the United States have been private enterprises, which separates them from most of their counterparts in the world. Canada's railroads, on the other hand, are a unique combination of a private firm and a crown corporation. Mexico chose early in its railroad history to follow the consensus of the global majority by nationalizing its railroads. In every case, however, serious and important relationships have developed between the private railroads and the public sector.

The historical role of government in the U.S.A. has been the regulation of the railroads' routes, service conditions, safety legislation and the general business environment. This was done at the state level by the Public Utilities Commissions and more preemptively by the federal government under the Interstate Commerce Commission. The Staggers Act brought welcome developments in federal deregulation, although the government's role in railroading was theoretically unaltered.

Government intervention is welcomed at times when markets fail and essential services become unprofitable. In some of our oldest cities, suburban rail transport has become necessary in order to keep the central city prosperous. Therefore, in times of trouble, instead of abandoning the railroad industry altogether, public entities have taken over the role of financing and operating existing commuter rail systems. Whether with their own people or under contract with private railroads, government-funded operating budgets are an accepted part of the urban scene. Examples of this trend include The Long Island Railroad, which became public as a whole and Metro North and New Jersey Transit are both publicly funded conglomerates of smaller private railroads. Chicago and Boston governing officials too have an effective working relationship with their local railroads.

Amtrak represents another kind of public involvement. Starting out as a safety net for services that most people thought should not die out, Amtrak has built a national network of long distance trains that link the country. The system has taken on a particularly significant role in high-profile niche markets, especially the densely traveled Northeast Corridor. A memorable *New York Times* headline a few years ago during a fare war on the Eastern Shuttle read: "Amtrak Winning Shuttle Wars."

All of these examples of successful public intervention seem to suggest that the government-private sector connection can efficiently tap the awesome potential of the railroad business. Each level of public involvement works well when it works to keep the railroad running on its own terms. In this way they work together and gain important social and economic advantages.

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Not only are new commuter lines developing, but also old lines are getting face-lifts. Metrolink in Los Angeles, Virginia Railway Express and MARC around Washington, DC, and TriRail in southeastern Florida already have started work on already-existing commuter lines. Seattle, Denver, Houston and Atlanta are getting ready to take the plunge. The new purpose of commuter rail is to satisfy an America that has become a nation of suburbanites, living and working in new places at the edge of our old cities. We now get business mail from places named Schaumberg, Irvine, Diamond Bar, Walnut Creek, Manassas and Gaithersburg. Since World War II, we have depended on the automobile for mobility in these places. Unfortunately, we seem to have come to the limits of what we can do with cars, or more exactly, the limits of where we can fit cars in. Traffic congestion is a constant in our suburban metropolitan centers and, in places like Los Angeles and Houston, we are choking to death on fumes as well. The Clean Air Act Amendments of 1990 made it unlawful to overlook the problem any more. Suddenly commuter rail is very alluring.

Rail rights-of-way turns out to be a previous opportunity. Here is a separate path, completely independent of the roadway system (except for grade crossings), that seems to offer passenger carrying capacity alongside the congested freeway. It is no accident the railroad is there. It was probably there first. The railroad surveyors opened the route a hundred years ago; they found the pass through the mountains, opened the route and established the flow of traffic. The freeway was probably an upgrade of the highway that followed the rail route into town. The logic of auto-based development brought the "new" city out along the interchanges. The congestion and air quality problems developed as the uncontrolled mix of traffic patterns and growth at the interchanges used up the capacity of the highway system. So it is not far to the bright idea: Why not look to the "underused" railroad to solve the traffic problem. But the suburban city as we now know it has developed without an orientation to the railroad. Even if the railroad is truly "underused" (not usually the case), it most often is not a perfect fit.

The task is to operate a public commuter rail system on the separate right-of-way, but in such a way that the existing private rail operations can continue. Here begins the clash of cultures.

I recommend to both railroaders and to public officials, elected or staff, the following operative practical virtue: Peaceful Coexistence, negatively expressed as avoiding the Lose/Lose situation, or more positively, Active Cooperation, embodying the Win/Win solution.

There are two sets of very good reasons to try this.

The first reason involves the benefits that both railroaders and public officials receive by making their collaboration work.

First, the railroad generally gets money they would not otherwise receive in other circumstances. Sometimes this is a very large amount. In Southern California, purchase of right-of-way and trackage rights approached $1 billion dollars. Beyond this, capital improvements to the rail line used for joint operations can contribute to greater efficiencies and smoother operations for the freight railroad as well as providing greater capacity on new commuter trains. Another considerable advantage is the ability for a railroad to divest itself of lines it no longer needs for its business and of the attendant maintenance obligations they represent. Finally, the new commuter operation can become a source of new business revenue for the railroad.

The benefits accrue to the government entity running the commuter service as well. Recognizing the currency of politics, a good commuter railroad translates into voter approval. The new commuter railroad provides what everyone is looking for so desperately: an alternative to the drive-alone automobile.

Rail transit is usually enormously expensive to start from scratch and who knows if it will work well enough to justify that effort and investment. A joint commuter rail operation avoids the prohibitive startup costs by using a line that is already there. And not to be underestimated, the public gets a system that may not be as glitzy as a new "monorail," but they will get a technology that has worked for over a hundred and fifty years.

NEW COMMUTER RAIL SYSTEMS: A RAIL SOLUTION TO A NON-RAIL PROBLEM

New cities turn to commuter rail not to have trains, but to clean up the air or to make the highways work. I am reminded of a state highway official who confronted me at a public meeting by asking: "Why should we taxpayers subsidize your rail operation up to 60 cents on the dollar?" (The legal farebox recovery goal in California is 40%.) I cleared my throat. "But wait," he says. "I'm on your side. If you can take 2% or 3% or 5% of the trips off my freeway, I won't have to spend billions on widening it or condemning your neighbors' houses for the right-of-way. I want you to succeed." So there it is in a nutshell.

Public entities are inviting themselves onto the railroad. There are plenty of good reasons for railroads to offer them hospitality. But to make the benefits flow well, it will be necessary to make sure both parties don't drive each other crazy. One of the best ways to prevent this is to understand the really different universes railroaders and politicians inhabit. Like any exploration of diversity, the first rule is to understand that each party is acting rather rationally and responding to cues and goals that are acceptable and reasonable in their climates. From this basis of assumed good will, one can then move on to explore the details of behavior each will exhibit in the many large and small matters they address. I wish to look at some of these behaviors by addressing the following categories: cultural context and scope of action, notions of time, financial dealings, constraints, accountability and success measurements.

Context and Scope

These, I think are the most fundamental areas in which railroaders and public officials differ and which need to be addressed to arrive at a mutual understanding. Context and scope operate in a distinct and fundamental manner within a culture.

The railroad tends to be a complete and for practical purposes self-contained environment. Rules and procedures govern the way the system operates and a sense of order prevails. Above all, the railroad strives to be orderly. A well functioning railroad is a prodigiously efficient mover of goods and people. It has harnessed the forces of physics and has developed methods that have withstood testing, have been proved to be safe and have led to great incremental productivity improvements.

Railroaders are specialists. They understand their business and their craft, and the interfaces between functions and departments. They have generally devoted a lifetime to the endeavor and are very good at what they do.

What they do is deliver transportation. This is a laudable activity, but it is important to recognize that it is only one of many in the world. There are many details, but there is one objective: to get the trains over the line. Everything focuses on achieving efficient operation.

Now I turn to the equal-time to the characteristics of the political arena.

As self-contained as a railroad can ever become, there still exists the complexity of all the influences pressing in one the public sector is hard to overstate. The people who live in this world are not just elected officials, but all levels of civil servants, appointed commissioners, career bureaucrats (a neutral term, but there is always that hint of disdain in the word, at least in the USA), even neighborhood activists. So this world appeals more to generalists, who need to be able to act with some confidence on many fronts, often at the same time. The public officials who are now looking at rail systems to solve real mobility and air quality challenges are not likely to be transportation professionals. They come to the issue from many fields, many of which are really as complicated in their own right as the railroad is. We, as voters, expect them to be able to react competently to health care, police, land use issues, economic development, gang violence in our cities, doing away with drug problems, keeping Social Security solvent, keeping the streets plowed and the trees trimmed, and making rules on the height of fences and shrubs. A misstep on

any one of these could derail their effectiveness on all the others. But now, especially at the metropolitan level, some will also need to have a working knowledge of how to make a railroad deliver mobility too.

Both these sketches are deliberately very broad and do not purport to describe any recognizable individuals. But if each group considers its own environment moderately normal, it is not hard to imagine how easily each could see the other's world as insane, or at least perverse.

Time

There is nothing quite so long as the five minutes waiting for a late bus, or nothing so short as the Fourth of July fireworks. Similarly general railroad time and political time seem to move at very different paces. The items on the railroad side of this list – continuity, longer horizons, a sense of history and institutional stability – emphasize again the self-contained nature of the enterprise. On the railroad, people remember. The nature of the business allows for a tradition that can be either written, or more likely, oral. Decisions are made on a technical scale. Equipment lasts a long time.

Tangent track can last 40 years and the rail can be cascaded for decades beyond that. Mechanics and technicians develop long term relationships with suppliers. Some roads are EMD properties, some are GE. Some may still want to buy Alcos.

While the railroad may seem like the Mississippi River, the public sector looks more like the ocean, with its constant ebb and flow. Watching the evening news shows how quickly terms of discussion transform. For better or worse, the urgency of the moment makes demands on political leaders, setting up the shorter time horizons and the election cycle as the metronome for public life. It is imperative to realize that many of the institutional arrangements for new commuter rail systems are very new themselves. In California, one of the great forces of innovation in government is the Joint Powers Authority, which allows several jurisdictions to join together to accomplish a very specific goal. These can be water districts, development agencies, even railroads. They are very focused, very flexible and usually resemble the creatures of the more established entities that formed them. The Southern California Regional Rail Authority, for example, is a combination of five counties which, each with their own mobility needs, have embraced the vision of making a single regional commuter railroad with the attendant economies of scale that come with acting together. While a permanent staff is building and operating the railroad, board members are appointed by the Transportation Commissions of each of the counties, and their members in turn are, for the most part, elected city council persons from individual jurisdictions. It is too harsh a judgement to call such arrangements precarious, but the strong local accountability built into such arrangements (by no means unusual in modern politics) calls for high-energy consensus building. I have been very impressed by the willingness to make things work that I have seen in this and other JPAs in the West. They are a breath of fresh air from the usual entrenched bureaucracies and and to a person with a railroad background, they can look exotic indeed.

Money

Both railroads and governments make plans. Each tries to evaluate accurately what the most pressing needs are and how to meet them with limited resources. Both need to decide what they can afford. Still, some differences arise. The railroad again makes its decision on revenue projections. If business is good, more capacity is needed, but not so much that it cannot be maintained in leaner times. The cost of labor and the rules for its use have much influence in decisions. Finally, the logic of the technology ultimately dictates what is best spent where on the the system.

On the political side, budgeting has become an extremely arcane discipline. What is in the budget, what is authorized, what is appropriated, what is actually spent are all separate concepts and numbers. Timing is also a crucial factor. Deadlines for applications and expiration of certain accounts are means for a government to control its internal processes, but they may not match the technical cycles of the rail projects for which a public agency wishes to pay. Some serious scheduling difficulties can arise from this issue. The "color" of money is a term for some of the conditions that must be observed in order to spend it on specific projects. State bonds in California have limit on how much "private benefit" can be brought

with borrowed money, a reasonable safeguard, but one that has to be calculated for the entire state. Putting together an entire program of public spending for several years of rail improvements can thus become a considerable technical feat.

Operating Constraints

This category describes the particular way railroaders and politicians say, "It can't be done." Or more positively expressed, what are the roadblocks that need to be overcome if we are to succeed?

For the railroad, constraints are generally technical. For capital goods, there is a certain amount of time needed to design facilities, order materials and have them delivered. Installation may take a great deal longer than instinct leads one to believe, because almost all line work has to be done under traffic. This is a very hard concept for lay persons to grasp. The extremely interlocked nature of railroad operations goes against all preconceptions of a population that has grown up with a car and can independently go almost anywhere at anytime without regard to the capacity of the roadway system (at least until recently). To use Southern California as an example, it is almost impossible for people to grasp what a feat it is to run 20 Amtrak trains, two Orange County commuter trains, several through freights and a couple of drill runs on an essentially single track railroad from Los Angeles to San Diego. It looks empty for half-hours at a time.

By the same token, so-called government gridlock should not be such a difficult concept to grasp either, when one considers the number of open-ended agreements that need to be put in place to permit laws to be passed. The operative word here is consensus, which means that many persons have to agree to get something done. What they agree on may not always be two related subjects. A classic pairing in New York State has traditionally been support for highways and roads upstate and subway fare subsidies downstate. As public discussion expands into even more subjects, such tradeoffs seem pretty straightforward and old-fashioned. It's when the railroad budget gets caught up with health care issues or aid to cities or even the general size of the budget itself, that the process can become less than transparent. But this is a legitimate constraint.

Another item to mention here is the large number of government jurisdictions. In 1980, there were something like 79,000 governments in the USA. The California JPAs I mentioned earlier are either part of this problem or part of its solution. No individual city or county could possibly oversee a whole railroad operation. Even our biggest cities rely on state help, even federal authority, to get the job done. Banding together is helping Southern California and other areas overcome the fragmentation of too many small cities trying to take on a big problem. At the same time, the JPAs need to interact with local cities with stations and with the agendas of the individual member governments, multiplying the number of players and the number of transactions and agreements to be maintained.

Accountability and Signs of Success

Railroads are responsible to shareholders and to customers, who are riders and shippers. Public entities are responsible to voters and taxpayers. That's what it says in the textbooks. Reality is more complicated than that. These terms can denote very abstract concepts and often cannot be adequately defined in any case, at least not enough to decide on a legitimate course of action.

A more useful discussion of accountable behavior might be the measures by which each culture defines its success. For the successful railroad as for the successful government, the main goal is to make sure the system works.

For the railroad, this means delivering goods and people on schedule. Making a reasonable profit in the process is also welcomed by management and owners. For a government, it means delivering the public services people have decided to demand in such a way that people use them and pay for them willingly. Consensus means that the users and the payers agree on this, even when they are not the same people.

The Cultural Bridges

The thrust of this presentation so far has been to point out the cultural differences, that can keep railroaders and public servants form working well together. My real message, however, is that both need to work well together and that neither can afford to shirk the challenge. The tools I propose are the generic concepts of communication and flexibility.

Communication Leads to Understanding

Such generalizations need a few practical expressions to start the process and keep it going. Every situation is going to be different. There are many diverse railroads and many diverse governments and many diverse mobility solutions that will be sought in the land. I offer a few simple steps that I trust will take a lot of effort for both parties to accomplish.

The first rule is to keep talking. This implies that one has started talking. Only by constant discussion of where each is and where each is trying to get will the process be accomplished. The greatest task will always be to see the situation from the point of view of the other. The insight to be gained is that the constraint the other faces is real, and not just imagined, in his system and his environment. (If, on the other hand, they discover together that the constraint is not real, then authentic progress has been made. Both systems have overcome a hurdle.) Trying to see how one system would meet the challenge posed in the other system is a tried and true mechanism for negotiation. It should be employed in this situation as well.

My experience has impressed on me the immense importance of talking about time. The temporal expectations of railroaders and public officials are very different and deeply imbedded. They color every discussion of what is reasonable or not. "When can you do this?" "When do I need to get this to you?" "How long does this process take?" are questions that belong in every conversation, if an objective is ever to be accomplished. Getting some agreement on schedule depends on both parties understanding what is realistic for each and why.

Finally, continue to talk.

Flexibility Leads to Action

The understanding that comes from intensive communication can result in creative attempts to make processes work for both parties.

The first effort is to help a partner over the difficulties that are part of his culture. Governments are often too small to deal with the great complexities of a functioning multi-state railroad. That same railroad, however, may be too big to be able even to identify a rewarding local opportunity, much less act on it within its own system and procedures.

A worthwhile endeavor is to establish on-the-scene coordination. The purpose here is to match two persons (or departments) of the same scale locally. They can relate to each other at the proper scale of the project, while at the same time understanding the purview of their own organizations. Each can talk to the other to accomplish mutual goals, but each knows his own organization and procedures and how to successfully deal with them.

Such action is a challenge for each of them and for their organizations. They are being called on to stretch the limits of their actions and to encourage, even prod, their organizations to do the same.

This is hard work. The fact that commuter railroads are growing in what were considered unlikely environments shows that many people from both cultures are making a major effort. They are showing that the large rewards to both railroads and public commuter authorities are real.

SINGLE TIE PUSH TEST FOR MEASUREMENT OF TRACK BUCKLING RESISTANCE

By: H. M. Lees, Jr.*

I'd like to talk this morning about track buckling, the Single Tie Push Test (STPT) and Burlington Northern's use of this method to investigate tie resistance.

The passage of trains and thermal expansion of the rail can create large compressive forces. The track structure is designed to safely distribute these forces to the subgrade. If the track structure fails to restrain rail movement, a track buckle or "sun kink" can result. In simple terms, a track buckle is a horizontal misalignment in the track, caused by the track structure's failure to restrain longitudinal rail forces.

There are two key elements which determine whether a track buckle occurs: the amount of compressive rail forces exerted, and the track structure's ability to restrain these forces. The tie-ballast interface is a critical location in resisting the longitudinal forces.

Measuring lateral resistance between a single tie and the ballast is important in determining whether track buckling might occur. The Single Tie Push Test was developed to perform these measurements.

The Single Tie Push Test equipment was originally developed by Foster Miller, Inc., a contractor to the U.S. Department of Transportation's Volpe National Transportation Systems Center.

The major mechanical components of STPT equipment include the STPT Rig or "head" and articulating jaw assembly which clamp the head to the timber tie. The major hydraulics components are a pump, hydraulic controls, including a pressure accumulator or reservoir, and a metering valve, hydraulic hoses, and piston in the STPT head. The electronic components include a pressure load cell and a load indicator box to display force. The original STPT equipment was designed to measure a maximum of 5,000 pounds of lateral force.

In the mid eighties, Burlington Northern Railroad initiated several track safety research projects. Track buckling or "sun-kinking" was a hot topic at that time. One BN project measured compressive forces from thermal rail expansion and the forces resulting from various train handling practices. The project investigated loaded and unloaded unit trains with and without dynamic brake and tractive effort at various speeds. The results were provided to the AAR and reported in October, 1990 in "AAR-Burlington Northern Railroad Train Handling Investigation Report.

While this research investigated the question of longitudinal rail forces and measured the magnitude of these forces, it did not address the second critical element in track buckling, namely the ability of the track to resist these forces. Measuring track lateral strength is critical if we wish to establish safe operating conditions and set appropriate maintenance practices.

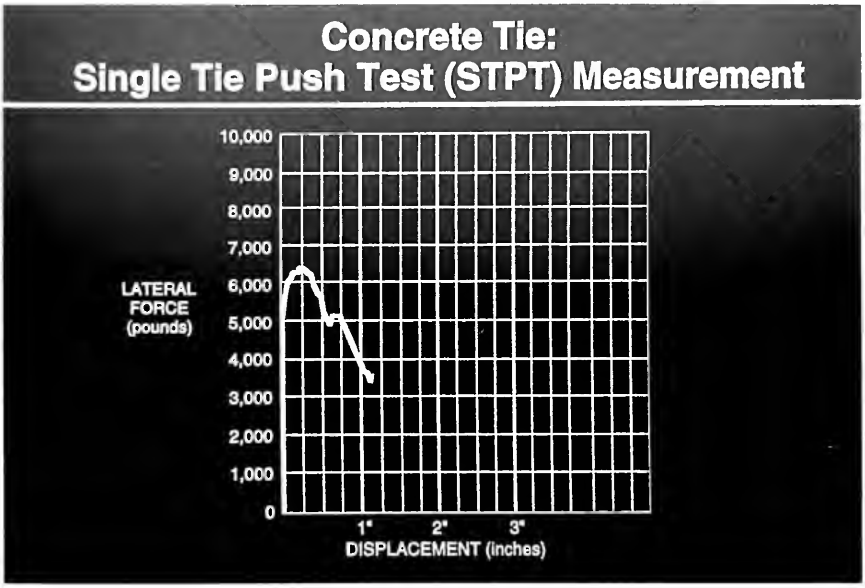
In 1986, Burlington Northern started installing concrete ties in our main line. Some maintenance personnel questioned whether concrete ties would provide adequate lateral strength due to their smooth sides. Ballast particles often embed in the side of timber ties, thus providing increased lateral resistance by mechanical means. In late 1988, Burlington Northern Railroad acquired Single Tie Push Test equipment to investigate the critical issue of track resistance strength at the tie-ballast interface on both timber and concrete ties. BN also intended to use STPT equipment to measure the performance of various types of ballast.

To use the Single Tie Push Test equipment on timber track, first the track spikes, tie plate, and anchors from both rails of the tie to be tested are removed. This forms a gap between the base of the rail and the top of the test tie. Next, the STPT head is placed on the test tie near the rail, and the jaw assembly with serrated edges on articulated arms is clamped on to the side of the tie. A hydraulic cylinder applies a horizontal, lateral force to the rail through a reaction block placed on the base of the rail. An electronic load cell located between the cylinder shaft and the reaction block measures the force applied to the rail. The STPT rig can be adjusted for various tie widths by moving the quick connect pins to adjacent holes. A hydraulic

*Manager, Track and Structures, Research and Development, Burlington Northern Railroad

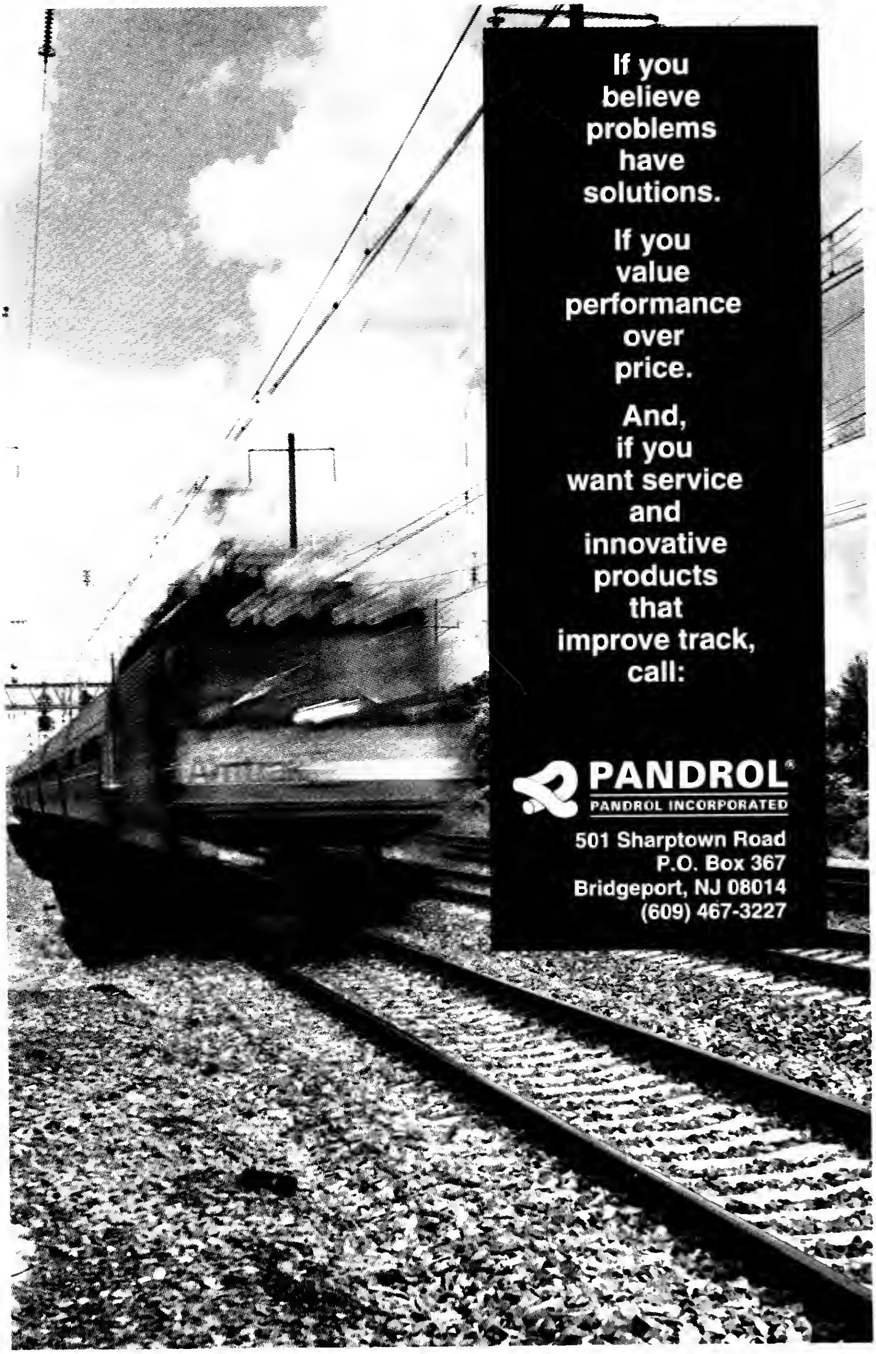
pump provides pressure to the hydraulic accumulator/reservoir, and a metering valve controls the cylinder ram extension speed to apply the load. The load indicator box records and displays the measured force, retaining the maximum or peak applied load. A mechanical dial gauge applied to the opposite rail measures the displacement, or an electronic string pot can be used to measure displacement. The electronic output from the force channel and the displacement measurement can be input directly into a plotter to provide an immediate plot of force vs. displacement. The STPT test is basically a nondestructive test, in that, it only disturbs one tie. To replace the tie to its original position, the tie can be moved back with track lining bars or the STPT head can be reversed to the opposite rail and used to push the tie back.

We needed to adapt the STPT equipment for use on concrete ties. To overcome the horizontal obstruction of the fastening shoulders, it was decided to use track jacks to elevate the base of the rail. For concrete ties with Pandrol fasteners, a Pandrol clip was cut in a "U" shape, and two short chains were welded to the clip. The other end of the chain connected tothe STPT head with bolts. The STPT head was elevated to the base of the rail by using threaded bolts which extended down to the top of the concrete tie. This initial "quick fix" proved successful and allowed measurement of concrete ties with Pandrol clips. This testing procedure was later calibrated using laboratory equipment to verify accuracy of measurements.



To adapt the STPT head to McKay fasteners, which are BN's current standard, we designed a metal plate for use on the field side shoulder. The center of the plate dimensions were the same as the base of the McKay spring clips. The plate was inserted in the shoulder and connected to two rods running underneath the base of the rail and was conneegted by chains to the STPT head.

To measure forces on concrete ties, one removes the fastening clips and insulating pads for a distance of approximately eight ties in either direction. Next, track jacks are installed in the crib and the rail is jacked up vertically until the base of the rail is slightly above the top of the shoulders. The STPT head is then placed on a tie one crib away from the track jack. This distance minimizes the effect of track jack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pressure on the adjacent tie. A horizontal force is then applied to the elevated rail to displace the tie; the lateral force and the displacement of the tie are recorded on a force displacement plot connected to the STPT equipment. With one hour track occupancy, four to six concrete tie STPT measurements can be made by an experienced crew.

Several improvements have been made to the STPT equipment. A portable generator was added for power. An electric-hydraulic pump now provides faster operation, and longer hydraulic hoses also increase operating speed. To measure strong tie resistance, the lateral force capacity of the equipment was increased from 5,000 to 13,000 pounds. The electric load cell was replaced with a hydraulic cell load, and the electric and hydraulic control equipment was combined in a single box. The addition of a force-displacement plotter, and the adaption of the STPT head to accommodate concrete ties has already been described. A portable PC-computer and printer now replace the plotter. The PC-printer is smaller and lighter than the old equipment, and permits recording, storage, and analysis of data and results.

Figure 2. Single Tie Push Test (STPT) Head

We have just completed construction of a new horizontal and vertical displacement measurement fixture. This fixture will measure tie displacement relative to the ground, rather than the opposite rail which might move during the applied load on concrete ties. This fixture is also designed to measure vertical tie displacement. Many believe vertical tie movement is an important factor in tie resistance. BN's 1988 track buckling training video described lateral tie ballast resistance force for unloaded track as 40% under the tie, 30% in the cribs, and 30% at the tie shoulder. With train loadings, the lateral resistance under the tie increased to 90%. While these figures are principally based on theories, it is easy to see the significance of tie uplift. The Transportation Systems Center has conducted numerous tests whichinclude STPT measurements with and without crib and shoulder ballast. (The results have not yet been published.)

While the STPT equipment has proven to be a useful and effective research tool, the equipment does have limitations. For example:

- The peak resistance value for a single tie often varies significantly from the average peak values, so measuring one tie is not adequate to establish typical tie resistance values.
- Since tie resistance will vary directly with, and increase with increased train traffic, it is important to know the history of ties to be tested. This is particularly important if the test objective is to make a relative comparison of two ballast types or different tie designs.
- The STPT measures the resistance of a single tie under a specific set of conditions, but it does not directly measure track resistance including fastener rigidity.
- The ballast condition can have a significant affect on peak STPT resistance values. High STPT resistance values do not necessarily mean a safe track. For example, fouled ballast or dry ballast at a track pumping site can yield high peak resistance values, but when this track is wet, the tie resistance will be low, and the track could be unsafe. Likewise, a measurement taken of a track with frozen ballast will not indicate appropriate tie strength of the same track under thawed conditions. A quick and simple probe of the ballast condition can eliminate many of these unknowns. The STPT is most effective in measuring clean uniform ballast.

While the STPT is a significant advancement in measuring tie-ballast strength, it is only one tool for determining track performance.

Having considered these limitations, let's concentrate on the STPT results. Measurements on weak ties produce a relatively low peak force which decreases slowly with further tie displacement. Weak ties (i.e. peak lateral force of 3500 pounds) are typical of the tie condition just after tamping. In comparison, strong ties develop high peak resistance within a short displacement, and the resistance decreases rapidly after the peak. The peak resistance on strong ties occurs between 1/4 and 1/2″ displacement. The Peak force on this test for a strong tie can be 6,500 pounds, which decreases to 4,000 pounds after only one inch of displacement. Weak ties typically provide 30 to 50% the resistance of strong ties. This general

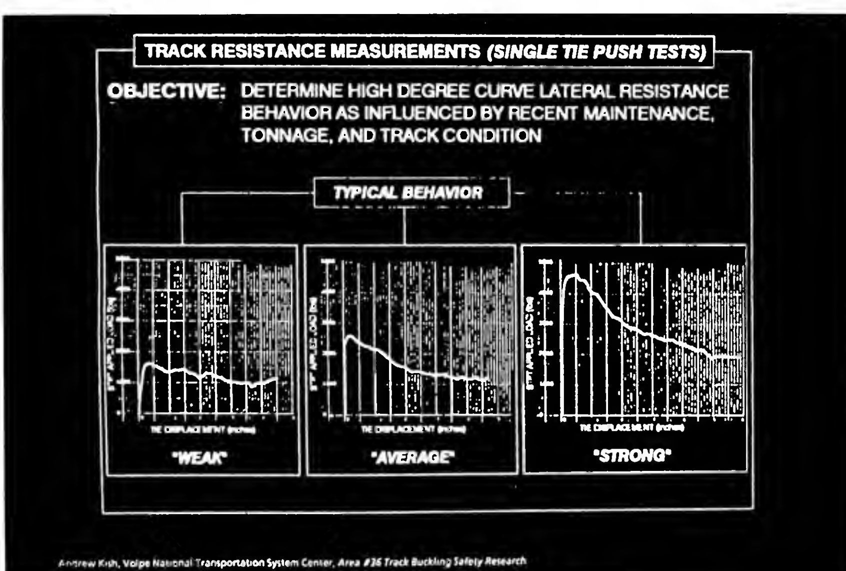


Figure 3. STPT for concrete ties with McKay fasteners.

relationship holds true for both timber and concrete ties. Concrete ties provide increased resistance compared to timber ties. Perhaps this is due to the concrete ties' rigid track structure, (higher track modulus) which results in less ballast disturbance. Increased rail traffic increases the tie-ballast resistance force. Figure 3, provided by the transportation systems center, shows typical tie resistance for weak, average and strong timber ties. On weak ties, peak resistance is 1,600 pounds, for average ties resistance is 2,600 pounds, and for strong ties the peak resistance is 4,600 pounds which occurs at 1/2" displacement. Tamping strong tie track can reduce peak resistance value from 4,600 lbs. to approximately 1,600 lbs., or 1/3 its previous resistance. The STPT Test Results are summarized in this slide. Weak ties provide 30-50% the resistance of strong ties. Peak resistance occurs at .25-.5 inch tie displacement. Concrete ties provide increased lateral resistance compared to timber ties. And tie resistance increases with increased train traffic.

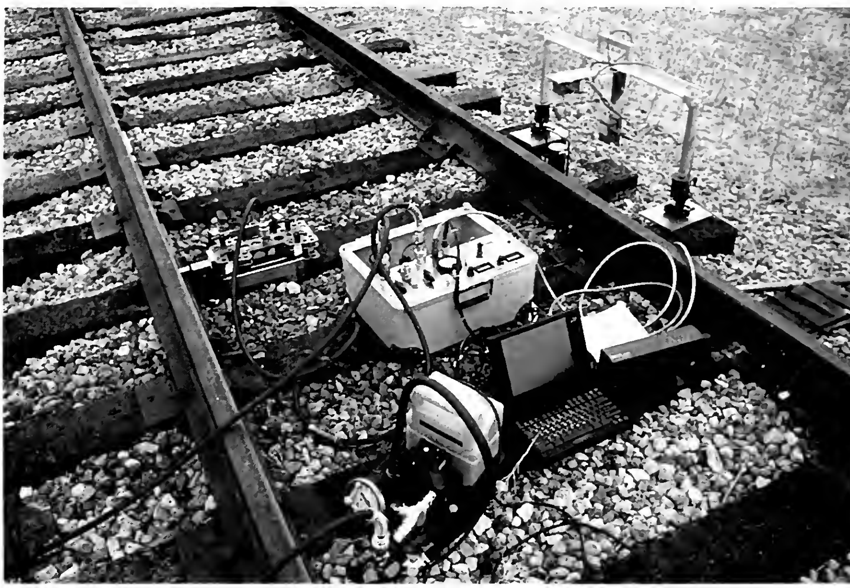


Figure 4. Improved STPT equipment with horizontal and vertical measurement fixture.

While BN's track buckling training and improved maintenance procedures have been successful in reducing track buckling incidents, I believe we need to know more about track resistance. This summer BN plans to conduct STPT measurements using the new displacement fixture, which will include tie uplift measurements. Additional measurement should be made to establish tie resistance in existing revenue service track, and to continue to monitor track performance in the future. Comparative tests should also be conducted to investigate the relative effect of various maintenance practices on tie resistance. These tests should include different tie designs, such as steel ties, and comparing ballast types to establish safe operating thresholds.

In conclusion, a number of people have helped in our work with the STPT equipment. I wish to acknowledge the help of Foster Miller, Inc., and to thank the AAR's Transportation Test Center personnel for their development and implementation of several of the STPT improvements, and for providing photographs for this presentation. Dr. Andy Kish, DOT's Transportation Systems Center has provided significant support and encouragement for this work. Finally, I'd like to thank Burlington Northern's management personnel for their support and commitment to developing innovative ideas in order to improve safety and operating performance.

PUBLISHED AS INFORMATION

COMMITTEE 1 - ROADWAY AND BALLAST

Chairman: W.L. Heide

Report of Subcommittee 10 - Geosynthetics

Subcommittee Chairman: T.R. Baas

GEOSYNTHETICS FOR USE IN RETAINING WALLS AND SLOPE STABILIZATION

INTRODUCTION

Geosynthetics may be used to reinforce soil structures such as retaining walls, slopes and embankments. The function of the geosynthetic reinforcement in Geosynthetic Reinforced Soil (GRS) walls and slopes is to provide tensile resistance to the driving forces caused by the dead weight of the soil and any applied surcharge. This resistance increases the factor of safety against failure and allows for the construction of steeper slopes and more economical retaining walls than would otherwise be possible using conventional gravity retaining structures. In particular, the advent of GRS slope design has given the engineer an additional method of constructing steep slopes at less cost than other types of retaining walls.

In general, GRS walls and slopes are constructed using alternating layers of horizontal reinforcement and compacted fill. The length and spacing of geosynthetic layers is a function of the geosynthetic type, type of soil backfill, phreatic surface, slope angle and height of the structure. For design, a geosynthetic reinforced structure is usually analyzed as a slope for slope angles up to 80 degrees and as a wall for slope angles from 80 degrees to vertical.

The purpose of this report is to provide a brief overview of GRS walls and slopes and describe how they may be used as economical retaining structures for railroads. This report is intended to parallel and complement a similar report prepared by Subcommittee 1 on Railway Applications for Mechanically Stabilized Embankments using Metallic Reinforcement.

APPLICATIONS

Typical applications for geosynthetic reinforced soil retaining walls and slopes include: vertical walls along property lines or natural barriers to increase land usage; bridge abutments for highway railroad grade separated crossings; grade separation for transportation right-a-ways in heavily congested areas; and vertical walls for dikes, jetties, quay walls, waterfront structures, architectural walls, military shelters, blast walls, noise barriers and boulder barriers. Figures 1 and 2 illustrate some typical applications for GRS walls and slopes respectively.

Railroad applications for GRS walls and slopes include:

1. Vertical retaining walls for increased land use of railway property, sidings and spur lines within limited right-of-way, bridge approach embankments in congested areas and bridge abutments. Several wall facing systems are available to fit the location and visibility of site.
2. Over-steepened slopes can be constructed instead of retaining walls where property boundaries or right-of-ways would not allow for construction of conventional slopes with side slopes constructed at the natural angle of repose of the soil. These structures can often be constructed at significantly less cost than any retaining wall.
3. Landslide repair can be done economically and effectively using geosynthetics. GRS slopes for landslide repair can be constructed using the on site (failed) soils with a much higher factor of safety than the original slope.

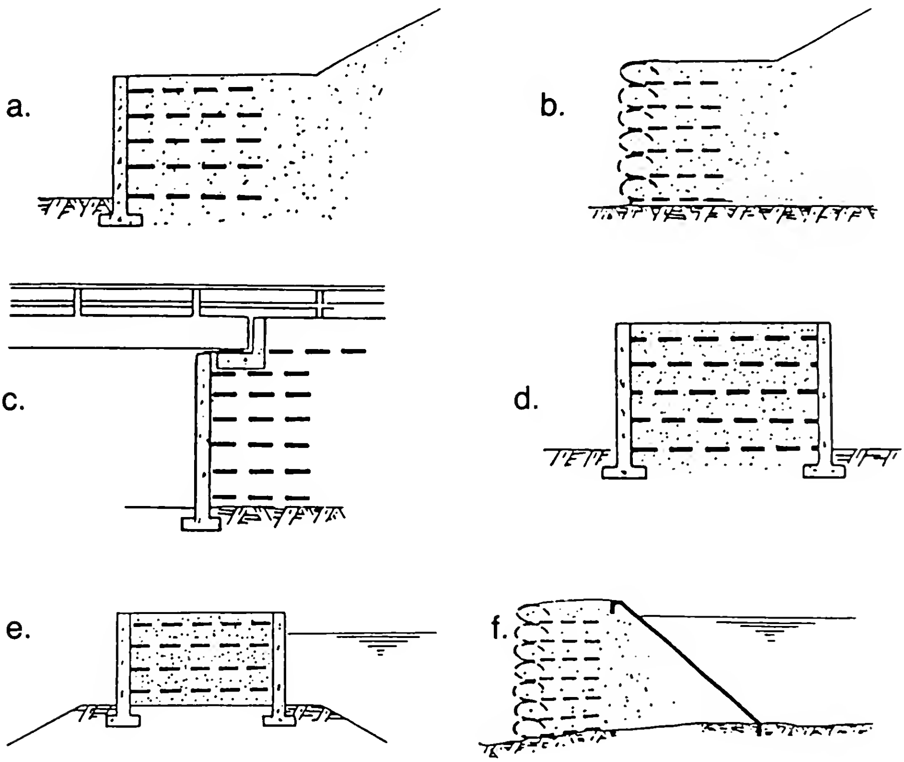


Figure 1. Reinforced Soil Walls: (a) Concrete faced retaining wall; (b) "Wrap-around" faced retaining wall; (c) Bridge abutment; (d) Dike or waterfront structure; (e) Dam extension; and (f) Dam.

4. Embankments over weak foundation soils can be economically constructed using geosynthetics without the need for excavation and removal of the weak soils. Such embankments can also be constructed with steeper side slopes to reduce the amount of fill required, thus, in most cases, reducing the total weight of the embankment and subsequent settlement.

TYPES OF GEOSYNTHETIC REINFORCING MATERIALS

The two most common geosynthetic materials used for soil stabilization are geogrids and geotextiles. A general description of the various types of geogrids and geotextiles is presented below:

Geogrids:

1. One type of geogrid is manufactured by drawing a perforated polymer sheet, in one or two directions. The polymer type, thickness of the polymer sheet, shape and size of the perforations and the amount of drawing dictates the size of the grid apertures and the reinforcing strength of the geogrid.

2. Other types of geogrids are manufactured by overlapping and bonding parallel polymer strands to similar strands at right angles to one another. The junctions may be heat bonded, welded or interwoven using the leno weaving technique.

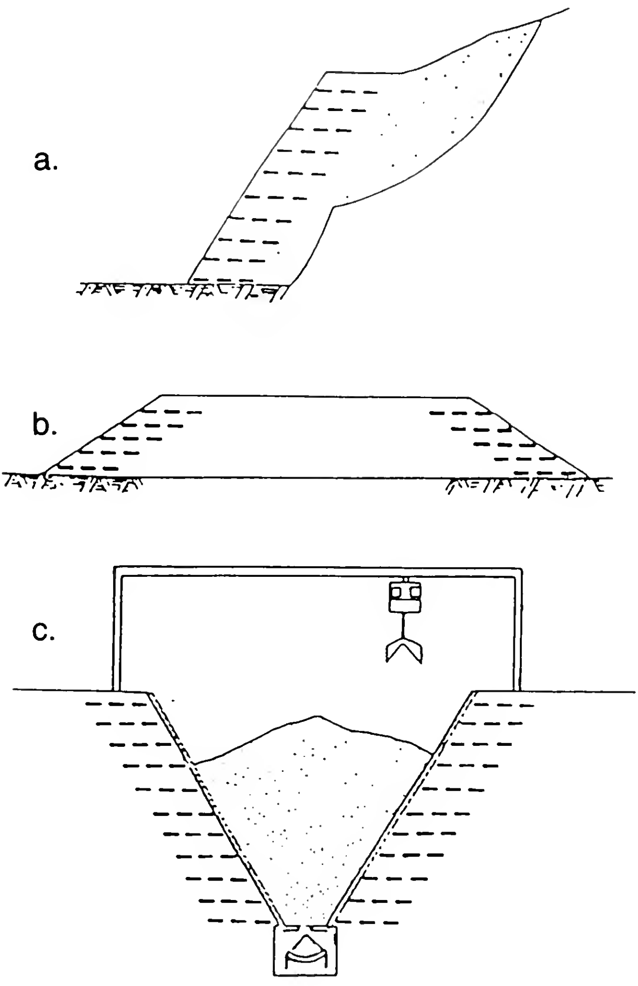


Figure 2. Reinforced Steep Slopes

(A) Landslide Repair
(b) Roadway Embankments
(c) Silo

Geotextiles:

There are three basic types of geotextiles used for civil engineering construction. They are woven, nonwoven and knitted, as described below.

Woven geotextiles are manufactured in a variety of sizes and weights, with a range of physical, mechanical and hydraulic properties, using conventional weaving machinery. Woven fabrics are bidirectional meaning that the fibers are oriented in two directions corresponding to the directions.

Nonwoven geotextiles are multidirectional, consisting of a random orientation of fibers. Several techniques are used to hold the random fibers in place, including heat bonding, chemical bonding, resin bonding and needle punching.

Knitted fabrics are manufactured by conventional knitting techniques.

Woven and nonwoven geotextiles are the most commonly used geotextiles in the United States. They are manufactured in a wide variety of sizes and weights, with corresponding variations in strength, tensile modulus and durability. The design of reinforced retaining walls with geotextiles requires special consideration of initial tangent modulus, long term creep and coefficient of friction between the geotextile and the retained backfill. Geotextiles are most commonly used for wraparound construction of low risk walls, typically up to 10 feet high, where some settlement due to elongation of reinforcement is allowable, such as reinforcement of embankment fills along temporary run around embankment roadways.

RETAINING WALLS

Definition:

A geosynthetic reinforced soil retaining wall is a prism of soil, reinforced with horizontal layers of geosynthetic reinforcement, and having a vertical or near vertical (up to 10 degree batter) wall face. A reinforced soil retaining wall consists of five major components, as shown in Figure 3. These include: 1) reinforced wall fill; 2) retained backfill behind the reinforced zone, 3) foundation soil, 4) geosynthetic reinforcing elements, and 5) wall facing elements.

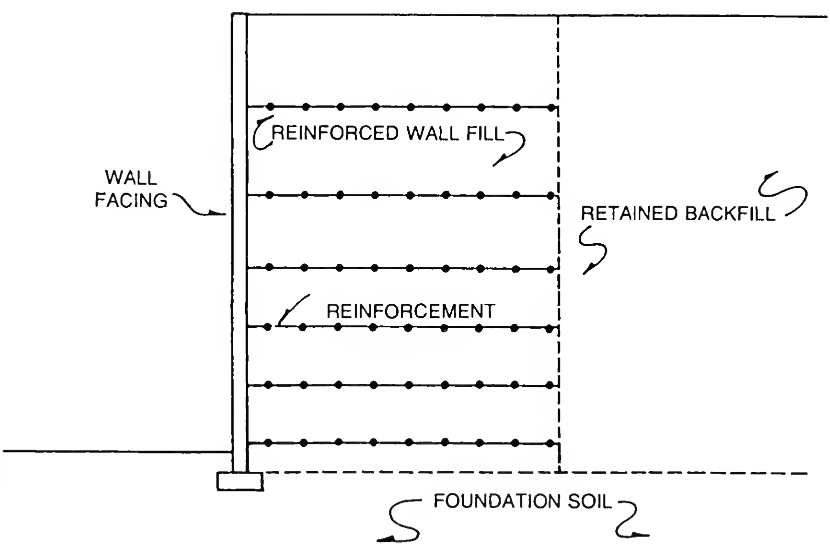


Figure 3. Definition of Terms

Wall Facing Materials:

Geosynthetic reinforced soil walls may be equipped with either concrete facing units, as shown in Figures 4 and 5, or wraparound construction, as shown in Figure 6. Other facing units that can be used include timbers, concrete blocks or bricks, wire mesh and gabion baskets.

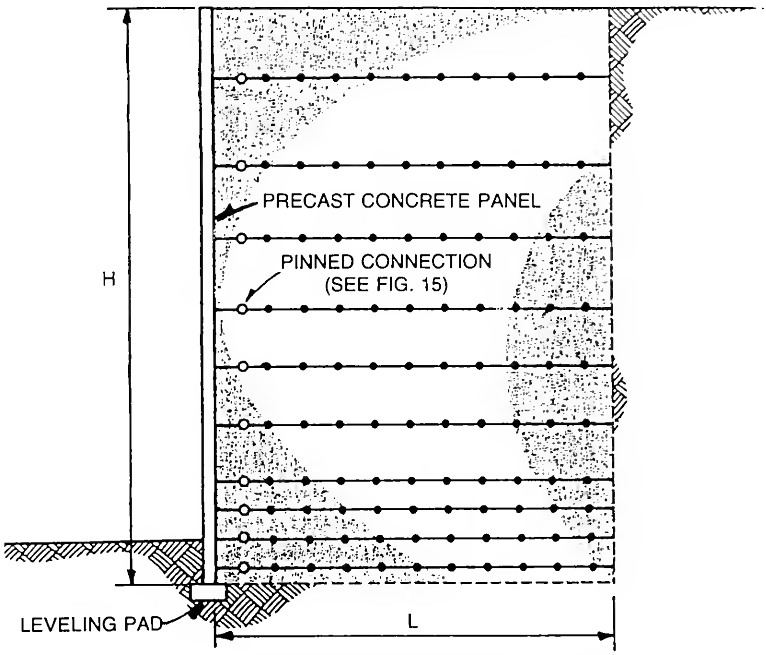


Figure 4. Full-Height Concrete Facing Panel

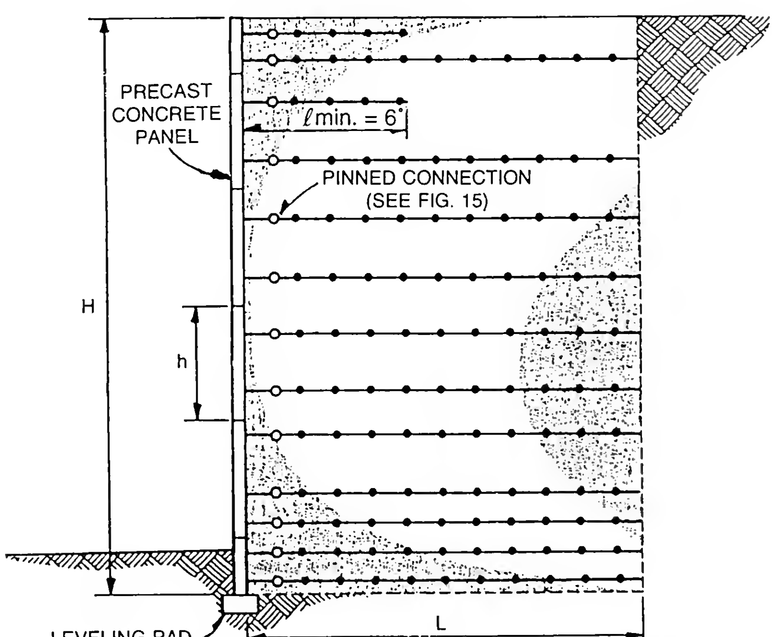
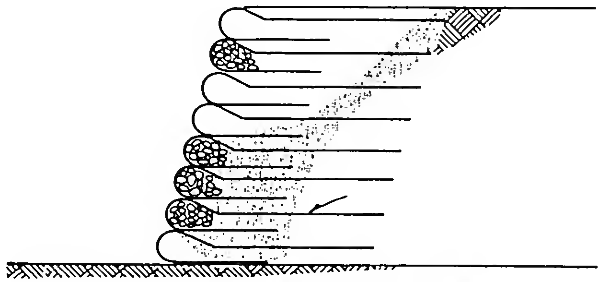
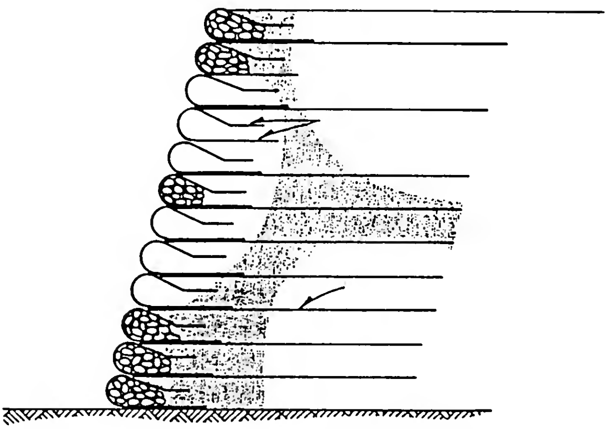


Figure 5. Typical Articulated Concrete Panel Faced Wall



a. CONTINUOUS WRAP



b. SEPARATE WRAP

Figure 6. Wrap-Around Wall

Articulated, incremental height precast concrete facing units are commonly used for geosynthetic reinforced soil wall structures. The precast units are usually 15 to 40 square feet in face area and a minimum of 5-1/2 inches thick. Retaining walls with articulated facing units are well suited for construction in confined areas since the units can be placed incrementally with equipment operating on top of the reinforced fill. Articulated walls are also advantageous since they can accommodate differential wall settlements, of up to approximately one percent.

Full-height concrete panels are also used to construct geosynthetic reinforced soil retaining walls. Full-height precast concrete panels are typically 8 to 10 feet wide and have been used to heights of approximately 25 feet. While full height panel walls are difficult to construct where limited space is available, they do offer the advantages of quicker erection, simple concrete forming, and optimization of geosynthetic placement over articulated concrete panels on some projects. However, differential settlement can be experienced with full height panels in walls over 25 feet in height.

Timber faced retaining walls and masonry block walls are used extensively for site development, roadway and architectural structures.

Gabion faced walls, with geosynthetic reinforced backfill are well suited for site development, erosion control and architectural applications. Since these systems are flexible and can tolerate relatively large post construction settlements, they are also suitable for railroad embankment construction where settlements are inevitable, but tolerable.

SLOPES

Definition:

A GRS slope is a soil slope or embankment that is reinforced with horizontal layers of a geosynthetic reinforcing material in order to increase its factor of safety against failure. Therefore, geosynthetic reinforcement allows a slope to be constructed steeper than would be possible without reinforcement. For example, unreinforced sand will be stable to slope angles between 30 and 35 degrees. The same sand, reinforced with geosynthetics can be designed for stability at slope angles of up to 90 degrees.

Slope Facing Materials:

Since GRS slopes can be constructed to a wide range of slope angles, it is necessary to consider some form of erosion protection or facing to retain the soil at the slope face. If the slope angle is less than 50 degrees, use of intermediate reinforcement (i.e. short strips of lightweight reinforcement placed at one (1) foot centers) and/or erosion control matting is usually sufficient to retain soil temporarily until vegetation can be established. For steeper slopes more permanent facing units are required. Examples include welded wire baskets, geosynthetic wrap-around or any wall facing system placed in a stepwise fashion such as timber ties, concrete blocks, gabions, etc. A turf reinforcement mat may be necessary to protect the soil face of steeper slopes.

Design

As with any soil structure, design of GRS walls and slopes requires identification of the typical soil properties of the foundation soil, retained backfill and the reinforced soil. In general, the required soil properties are soil unit weights, angle of internal friction and cohesion. Effective stress parameters should be used for long term stability analysis while short term stability of embankments over soft ground should be checked with total stress parameters. In addition, it is necessary to accurately determine several key design parameters for the geosynthetic reinforcing material under consideration. The following recommendations for selection of geosynthetic design parameters have been developed for highway retaining structures by Task Force 27 of AASHTO-AGC-ARTBA, 1989. These design guidelines have also been accepted by the Federal Highway Administration as acceptable guidelines for the design of GRS walls and slopes.

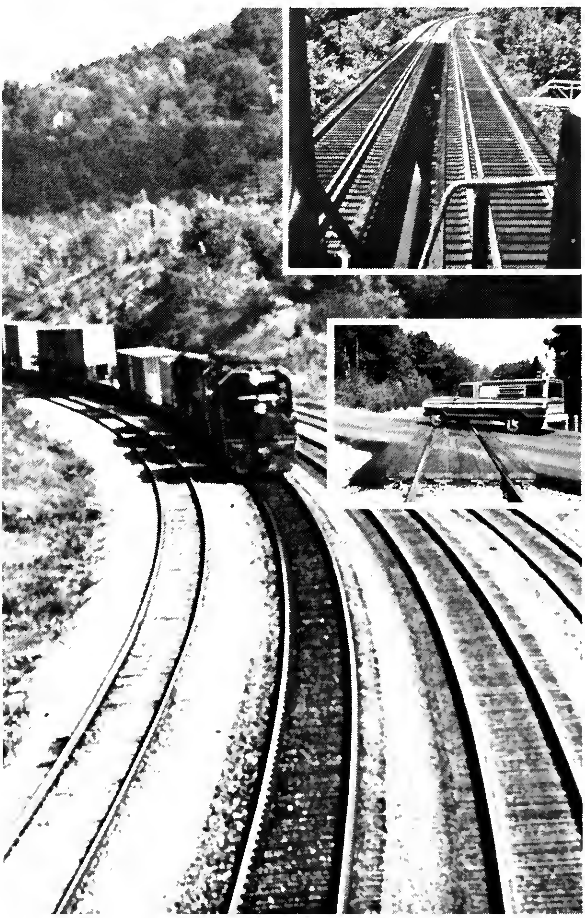
Geosynthetic Design Parameters:

The proper determination of the long term design strength of a geosynthetic reinforcement (GRI) can be made using the guidelines of Task Force 27 and the Geosynthetic Research Institute Standards of Practice as listed below:

GRI-GG4A - LTDS Determination for Stiff Geogrids
GRI-GG4B - LTDS Determination for Flexible Geogrids
GRI-GG5 - Geogrid Pullout Behavior

Task Force 27 and the GRI Standards of Practice set forth the limiting design factors in the determination of the long term design strength of a geosynthetic reinforcement. These limiting factors are addressed through the use of Factors of Safety or full default values when necessary. These limiting design factors are as follows:

FS_{CR} - Creep Deformation
FS_{ID} - Installation Damage
FS_{CD} - Chemical Degradation
FS_{BD} - Biological Degradation
FS_{JCT} - Junction Strength
FS_{JNT} - Joints and Seams
FS_{DU} - Design Uncertainties



Task Force 27 and GRI require that these partial Factors of Safety be determined and documented through site specific testing on full representative product samples. If site specific testing is not available or not applicable for the use of the partial Factors of Safety then the full default values listed in Table 1 will be used in the determination of the allowable long term design strength of a particular geosynthetic reinforcement.

Table 1. Default Values for Geogrids for Various Partial Factors of Safety

Application	FS_{ID}	FS_{CR}	FS_{CD}	FS_{BD}	FS_{JCT}*	FS_{JNT}
Embankments	1.4	3.5	1.4	1.3	3.0	2.0
Slopes	1.4	3.5	1.4	1.3	3.0	2.0
Retaining Walls	1.4	3.5	1.4	1.3	3.0	2.0
Bearing Capacity	1.5	3.5	1.6	1.3	3.0	2.0

*To be used if junction strength tests are not utilized in determining the other partial Factors of Safety values. If junction strength tests are utilized this value is 1.0 for all applications.

The long term design strength of a geosynthetic reinforcement is calculated using the formula in Table 2.

$$T_{allow} = \frac{T_{ult}}{FS_{ID} \times FS_{CR} \times FS_{CD} \times FS_{BD} \times FS_{JCT*} \times FS_{JNT} \times FS_{DU}}$$

where

FS_{ID}	=	partial Factor of Safety for installation damage
FS_{CR}	=	partial Factor of Safety for creep deformation
FS_{CD}	=	partial Factor of Safety for chemical degradation
FS_{BD}	=	partial Factor of Safety for biological degradation
FS_{JCT*}	=	partial Factor of Safety junction strength
FS_{JNT}	=	partial Factor of Safety for joints (seam and connections)
FS_{DU}	=	partial Factor of Safety for design uncertainties
T_{ult}	=	ultimate strength (kN/m or lb/ft)
T_{allow}	=	allowable strength (kN/m or lb/ft)

Foundation Soils:

As with the design of any structure, a careful analysis of the foundation soils must be carried out to determine bearing capacity and external stability of the structure to be built.

For external stability of GRS walls, the reinforced soil prism is considered as a large gravity structure and, as such, is analyzed for sliding, overturning, tilting or bearing capacity failure, and global or overall slope stability.

Sliding failure refers to the action of the entire reinforced wall fill block or mass being driven outwards by the lateral thrust of the retained backfill. The Factor of Safety, F.S., against sliding is defined as the resisting frictional force at the base of the wall divided by this lateral thrust.

Overturning failure is based upon the assumption that the reinforced soil mass behaves as a rigid body which resists the overturning forces exerted by the lateral thrust of the retained backfill. The Factor of Safety for overturning is defined as the resisting moment generated by the reinforced soil mass, about the wall toe, divided by the overturning moment due to the lateral thrust.

The Factor of Safety against bearing capacity failure of the foundation may be estimated using a Meyerhoff type of pressure distribution. A uniform bearing pressure is assumed to exist over a length equal to (L-2e), where (e) equals eccentricity of the bearing pressure resultant from the vertical centerline of the wall fill. The Factor of Safety for bearing failure is equal to the ultimate bearing capacity divided by the applied bearing pressure.

The overall or global stability, of which the reinforced zone is only a part, should be checked using routine slope stability analysis methods. Slope stability safety factors ranging from 1.3 to 1.5 or greater are typical of standard engineering practice.

The external stability of GRS slopes soils should be analyzed for possible failure surfaces behind the reinforced soil mass and through the foundation soil.

Internal Stability - Walls:

For internal stability, the reinforced soil retaining wall must be coherent and self supporting under its own weight and any externally applied forces. The geosynthetic reinforcement must be selected and spaced to preclude tension rupture and to prevent pullout from the soil mass beyond the assumed failure surface. Since geosynthetic reinforcements are extensible and require a larger tensile strain under working conditions than the horizontal extension required to generate active lateral earth pressures, the tieback wedge method of analysis is typically used for analysis of geosynthetic reinforced soil retaining walls. With this method it is assumed that full shear strength of the reinforced fill is mobilized and active lateral earth pressures are developed. These pressures must then be resisted by the reinforcement tensile force. The wall kinematic mechanism is assumed to be rotation of the wall about a hinged toe. The assumed Ranking failure plane is defined by a straight line passing through the wall toe and oriented at an angle of $45° - (\phi/2)$ from the vertical.

The tensile force per unit width in the reinforcement is calculated as a function of the vertical stress induced by gravity, uniform normal surcharges and active thrust from the retained fill, multiplied by the active earth pressure coefficient (K_a). The required reinforcement tension to prevent rupture shall be less than or equal to the allowable reinforcement tension as determined for the geosynthetic under consideration.

Internal Stability - Slopes:

Internal failure of GRS slopes may result from reinforcement rupture and/or reinforcement pullout. Therefore, the long term design strength and pullout resistance of the reinforcement for the particular type of soil to be used should be determined using the methods described above. Limit equilibrium methods are typically used to evaluate GRS slopes by assuming that the geosynthetic reinforcement provides additional resisting forces to increase the Factor of Safety of the slope. Well known slope stability analysis methods, such as Bishop's Modified Method of Morgenstern and Price's Method have been simply modified to include geosynthetic reinforcement as additional resisting forces.

Although the use of GRS walls and slopes has grown rapidly in the last several years, there are many practicing engineers, including railway engineers, who are unfamiliar with the technology review and evaluation criteria and design methodology that is required to properly utilize these soil reinforcement systems in the most economical and practical fashion.

A complete evaluation of the site specific soil/project/site parameters is required to properly use these soil retention solutions. This requirement cannot be overlooked and must be addressed by the engineer or his qualified representative.

Three (3) practical approaches are available to the engineer to properly apply and effectively use this technology.

1. Upon completion of a thorough site investigation the engineer may require the manufacturer(s) to provide design, materials, and site assistance within the scope of the project objectives. Should this approach be taken, the engineer or his qualified representative should review the submitted design information to ensure conformance with the project specifications and objectives. The FHWA offers guidance for this approach in the following document; *"Guidelines For Geosynthetic Mechanically Reinforced Earth Slopes"*.

2. Contract with an outside engineering and/or design build firm to act as the engineers representative. In this case the outside engineering firm is responsible for the site investigation, design, and site assistance requirements.
3. Provide an in-house design. Should this option be taken, the guidance required in this approach can be obtained from the AASHTO-AGC-ARTBA Task Force 27 Soil Reinforcement Guidelines, Geosynthetic Research Institute standards (GRI), the FHWA Geosynthetic Mechanically Reinforced Earth Slope Design Guidelines.

CONSTRUCTION

Construction of GRS walls and slopes requires only slight modifications to conventional methods of constructing retaining walls and slopes. The primary difference is that horizontal layers of geosynthetic reinforcement be placed in the compacted soil at predetermined elevations. The length of the reinforcement layers is also determined by design. For GRS walls, it is also necessary to connect the reinforcement to the wall facing unit and apply tension to the geosynthetic to ensure that no slack exists when it is covered with the next lift of backfill. A variety of connection and tensioning methods are available to best suit the type of structure and type of facing unit used.

GRS Wall Construction:

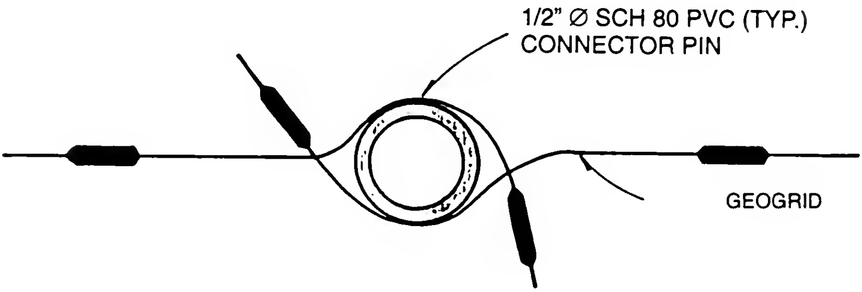
Prepare the foundation area by conventional methods of grading and compaction, removing any unsuitable material and replacing it with compacted backfill. For concrete or other relatively rigid facing types, construct an unreinforced concrete leveling pad, typically 6 inches thick by 12 inches wide.

For concrete facing units such as incremental and full height panels, geosynthetic strips may be cast into the panels and connected to the main reinforcement strips during construction on the wall. The facing panels should then be placed on the leveling pad followed by placement and compaction of the backfill up to the elevation of the first layer of geosynthetic reinforcement. Full height panels will require bracing in front of the wall during placement of the backfill. Connect the main reinforcement strip to the wall panel. For geogrids with high junction strengths, this connection can be made by sliding a polymer connecting bar through the end apertures of the precast strip and the main reinforcing strip simultaneously (Figure 7). If a reinforcing strip is not precast into the panel, the main reinforcement can be attached to a steel bar, which in turn is fastened to the wall panel. The geosynthetic strip should then be tensioned to remove all slack. Subsequent lifts of compacted backfill and geosynthetic reinforcing strips should be placed as described above. Placement of filters to retain soil along joints and weep holes to drain water from the retained backfill should be done according to conventional practice.

As stated previously, other wall facing types include concrete blocks, timber ties, wire basket and gabions and geosynthetic wraparound (Figure 6). In general, geosynthetic reinforcement is placed between lifts of these facing units at predetermined elevations. Connection of the geosynthetic to the facing unit may be achieved simply by the weight of the facing unit on top of the geosynthetic or, by design, may require additional fastening measures such as mortaring, tie connections or pin connections protruding through the reinforcement. If the geosynthetic is to be facing using the wraparound method of construction, temporary form work is required to enable compaction of the backfill at the wall face prior to wrapping the geosynthetic into the next lift of compacted backfill.

GRS Slope Construction:

Construction of GRS slopes to less than a 50 degree slope angle do not necessarily require any type of facing other than erosion protection at the face. Therefore, a geosynthetic reinforced slope is constructed in the same manner as a conventional slope with the inclusion of the reinforcement at the prescribed elevations. Placement of the geosynthetic does not require tensioning, provided that it is laid down flat and free of wrinkles. For slopes steeper than the angle of internal friction of the soil, secondary layers of reinforcement are usually placed along the face at frequent vertical intervals to facilitate compaction of the fill along the slope face.



NOT TO SCALE

Figure 7. Pinned Connection

For slopes greater than 50 degrees, wraparound construction or some form of facing is required. Connection of the facing unit to the geosynthetic reinforcement is required as described above.

RAILROAD APPLICATIONS

Railroad applications for GRS walls and slopes include track embankments, bridge approaches, expansion of existing facilities over sloping ground and landslide repair. Examples of geogrid reinforced wall and slope projects are listed in Tables 1 and 2.

There are several advantages to GRS walls and slopes in these railway applications. These include:

1. Nonmetallic reinforcement, which is noncorrosive and assures adequate reinforcement for the design life of the structure.
2. Nonmetallic reinforcement does not interfere with track circuits for signal devices.
3. GRS walls and slopes can tolerate settlements without damage. Therefore, structures that can tolerate settlements without damage such as railway embankments over compressible soils can be constructed at a much lower cost by eliminating the need for removal of the compressible soil.
4. Construction does not require any specialized equipment or labor, enabling work to be carried out by railroad work crews.
5. Use of on-site materials, eliminating the need for transporting granular materials to the site.

Possible disadvantages that need to be addressed on a job specific basis are as follows:

1. Access to utilities within or below a reinforced soil. Since it is not recommended to cut through the reinforcement, it would be necessary to move the utility around the reinforced soil zone or encase it in concrete or steel near the surface.
2. Affects of chemical spills on the geosynthetic reinforced soil. Geosynthetics are resistant to most chemicals and should not be significantly affected by minor spills or leaks. The chemical resistivity of the polymer used for the geosynthetic can easily be checked using published charts. In the event of a major spill, it is likely that the entire structure will have to be reconstructed to remove the contaminated soil and be reconstructed.

SUMMARY AND RECOMMENDATIONS

The objectives of this paper has been to increase awareness of the potential uses and cost advantages of GRS walls and slopes for railway application. While existing design methods are suitable for many railroad applications, it is proposed to investigate, through further committee activity, the effects of railway loadings and the resulting dynamic loads on GRS structures and prepare specific design and construction guidelines to be incorporated into future revisions of the AREA manual.

Table 1. Railroad—Walls

Project	Date	Location	Owner Designer	Contractor	Maximum Height	Face Treatment	Notes
Railcar Loading Dock		Salt Lake City, UT	Union Pacific Railroad/P.R.C.			Wrap	
CN Boston Bar	1983	Boston Bar, B.C.	Canadian National Railways/EBA Engineering	View Construction	20′	Shotcrete	
CRT Sait Hill	1986	Calgary, Alta.	City of Calgary, Light Rail Transit/Simpson, Lester, Goodrich	Geocrete	49′	Concrete	

Table 2. Railroads—Slopes/Embankments

Project	Date	Location	Owner Designer	Contractor	Maximum Height	Slope Angle	Face Treatment	Product	Comments
CP Rail, Landslide Repair	12/82	Waterdown, Ontario	Canadian Pacific Railways	Godfryd Contracting	30′	1.1	Wrap	UX1200 (2200sy)	Slide endangered rail line on Niagara escarpment. Repair completed in 12 days.
MARTA, South Yard	12/85	Atlanta, GA	Metropolitan Atlanta Rapid Transit Authority/Parsons, Brinkerhoff, Quade and Douglas	APAC-Georgia	21′	1.3H:IV	Mat		Steepened berm at toe of slope for landslide correction.
Seaboard RR	4/85	Tennessee	Seaboard System RR/Geologic Associate		40′	0.5H:1V			
AT & SF Railway		Owasso, OK	Santa Fe Railroad						Steepened berm at toe of slope for landslide correction.
CN Rail	9/84	Thunder Bay, Ontario	Canadian National Railroad		38′	2H:1V	Grass	BX1200	Embankment slide repair.
Southern Pacific	'87	Converse, Texas	Southern Pacific	Pat Baker Co.	18′	2H:1V	Grass	BX1100	Embankment slide repair.
AT & SF Railway	7/87	Menary, TX	Santa Fe Railroad					BX1200	Reinforcement to increase embankment stability.
Brock Rd., Hwy. 401 Interchange	8/86	Pickering, Ontario	City of Toronto, Go Transit and Ontario MTC			1.5H:1V	Net seeding	BX1200	Slide Repair. Steepend slope for Go Transit Rail Line. Replaced retaining wall.

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COMMITTEE 8 - CONCRETE STRUCTURES AND FOUNDATIONS

Chairman: G. W. Cooke

PRODUCING DURABLE ECONOMIC CONCRETE

Concrete is easy to make, however, making durable economical concrete to suit service requirements is more difficult. Although highly durable concrete, subjected to severe service conditions, can be made, (e.g. ferro-concrete hulled boats) there is a perception that today's concrete is less durable than before.

Most of the problems of non-durable concrete arise from not understanding the properties of concrete, lack of adequate production controls and the desire to produce "economical" concrete.

The biggest effect on concrete has been changes in the methods of making cements over the last 50 years. Modern cements are more finely ground and have a slightly different composition than cements of earlier times. This has permitted concretes to achieve higher strengths in 28 days (the usual criteria for specifying concrete), using less cement than earlier concretes. With the same water content, but reduced cement, concretes with a specified strength now have higher water-cement ratios (W/C) than in years before. As a result, modern concrete accepted on the 28 day minimum strength criterion, will be more porous and consequently, more susceptible to deterioration than the older concretes with the same 28 day strength.

With higher strength concrete commonly available, designers can now design thin members and will specify little cover over the reinforcment. This contrasts with old-time designs where weaker concrete (as measured by the 28 day acceptance criterion) meant members were considerably more massive and had routinely more cover on the reinforcing. The thin cover and greater porosity of modern concrete makes the reinforcing steel more susceptible to corrosion.

Finally, the production of concrete uses a significant amount of expensive field labor, and this, combined with the high cost of materials, creates pressure to place, finish and put into service these structures as quickly as possible. This economic strain increases the possibility that there will be questionable placing and finishing practices and the concrete will experience premature shrinkage cracks as a result of being inadequately cured.

Durable economic concrete is made through a three-step interactive process. First, it is necessary to understand what the deteriorating mechanisms are. Second, it is necessary to understand how those functions responsible for controlling the deteriorating mechanisms play their role in the process of producing durable concrete. Third, it is necessary to understand that there are economic trade-offs between paying high initial costs and paying higher long-term maintenance costs.

A. Deteriorating Mechanisms of Concrete:

The following are common deteriorating mechanisms and the methods of reducing and/or protecting against their effects.

Deteriorating mechanism	Protection
1) Freeze-thaw action:	—use appropriate amount of cement for expected exposure condition —use suitable low water-cement ratio —provide a suitable degree of air-entrainment —prevent the loss of moisture from concrete while curing —design the structure to minimize exposure to moisture using geometrical details which will prevent water from collecting on, or will direct water away from the structure —protect the structure against moisture by using membranes or sealants —use good construction practices such as avoiding over-finishing or avoiding the addition of water to aid finishing. (This brings water to the surface and increases W/C ratio there).

2) Chemical Attack:	
a) Attach on the cement	
i) Sulphate	—use resistant cement —use low water-cement ratio —use air-entrainment to permit reduction of water-cement ratio —replace part of the cement with pozzolan
ii) Acid	—use low water-cement ratio —use membrane or coatings
b) Chemical attack on the aggregates (alkali-silica reaction alkali-carbonate reaction or cement-aggregate reactions)	—avoid use of susceptible aggregates —use low alkali cement —replace part of cement with pozzolan or fly-ash —reduce exposure to moisture by using sealants or membranes —use less chloride de-icing salts
c) Chemical attack on reinforcing steel	—use low-water cement ratio (W/C) —use greater proportion of cement —use air-entrainment or superplasticizers to improve workability while reducing W/C ratios —provide good concrete cover on steel to maintain alkaline pH around bars —avoid details which will permit water to collect —provide good drainage —use smaller spacing of bars to avoid creating cracks between bars from shrinkage or temperature effects —reduce exposure to chlorides either in aggregates during construction or de-icing salts while in service —avoid exposure to stray electrical currents —use low slump dense concrete overlay, or styrene-butadien latex-modified concrete —use epoxy coated bars —use waterproofing membrane
3) Mechanical Wear: (abrasion and erosion)	—use abrasion resistant aggregate —use low W/C concrete with well-graded aggregate and low slump —use higher strength concrete —avoid finishing too soon —reduce surface W/C ratio by vacuum dewatering (slabs) —use proper curing procedures, keeping concrete continuously moist, or by using curing compounds —avoid exposing curing concrete to high levels of carbon dioxide

B. Functions Responsible for Controlling Deteriorating Mechanisms:

The production of durable concrete falls under five major but distinct functions. Each function, which in this context means an entity or person with specific duties and responsibilities, must be fully cognizant of the deteriorating mechanisms which are under their control and exercise their responsibilities.

1) Designer:

The designer is responsible for good design details and proper specifications for the expected service condition, and has the prime responsibility for ensuring that the required durability will be achieved.

2) Concrete Supplier:

The concrete supplier is responsible for good mix design, producing good concrete and usually transportation to the site.

3) Concrete Worker:

The concrete worker is responsible for good field practice in the handling, finishing and curing of concrete.

4) Inspector:

The inspector is responsible for good quality control to ensure the concrete meets design specifications, and is placed, finished and cured in accordance with good construction practices.

5) Owner:

The owner is responsible for ensuring the structure is used for the purpose intended and it is properly maintained during its entire service life.

The flow chart in Figure 1 summarizes the processes in producing durable, economical concrete. Failure within any of the five major functions will result in less than durable concrete.

C. Economic Considerations:

Durable economic concrete is concrete which achieves the expected life for the minimum discounted cost of initial capital outlay and future maintenance costs. When building a new structure, the owner has several options. It can initially spend more for concrete protection increasing its immediate cash outflow but reducing its future outflows due to reduced maintenance. It only reaps the benefits of the protection over a long period of time. Alternatively, it can pay less for initial concrete protection, but incur future higher maintenance costs. Factors which may increase the tendency to reduce initial cash outflows for concrete protection:

1) High financing costs adversely affect the viability or profitability of the project.

2) Future maintenance costs can be passed on to others.

3) The resale price for more cheaply-made structures can be equal to better-made structures, increasing profitability, but the quality difference is difficult to appreciate, especially in newer structures.

Note that there are often tax incentives to sell a structure, especially a building, after depreciation has been expended.

4) There is a high rate of obsolescence.

In summary, it is important to realize that production of durable economic concrete requires a combination of recognizing the deteriorating mechanisms, controlling or protecting against these mechanisms and a willingness to pay for the initial protection against these mechanisms. There is no doubt that, with today's understanding of concrete, highly durable concrete can be produced. It is just a matter of spending the money and maintaining careful control over the entire process. The old adage "you get what you pay for" is ever true when producing durable concrete.

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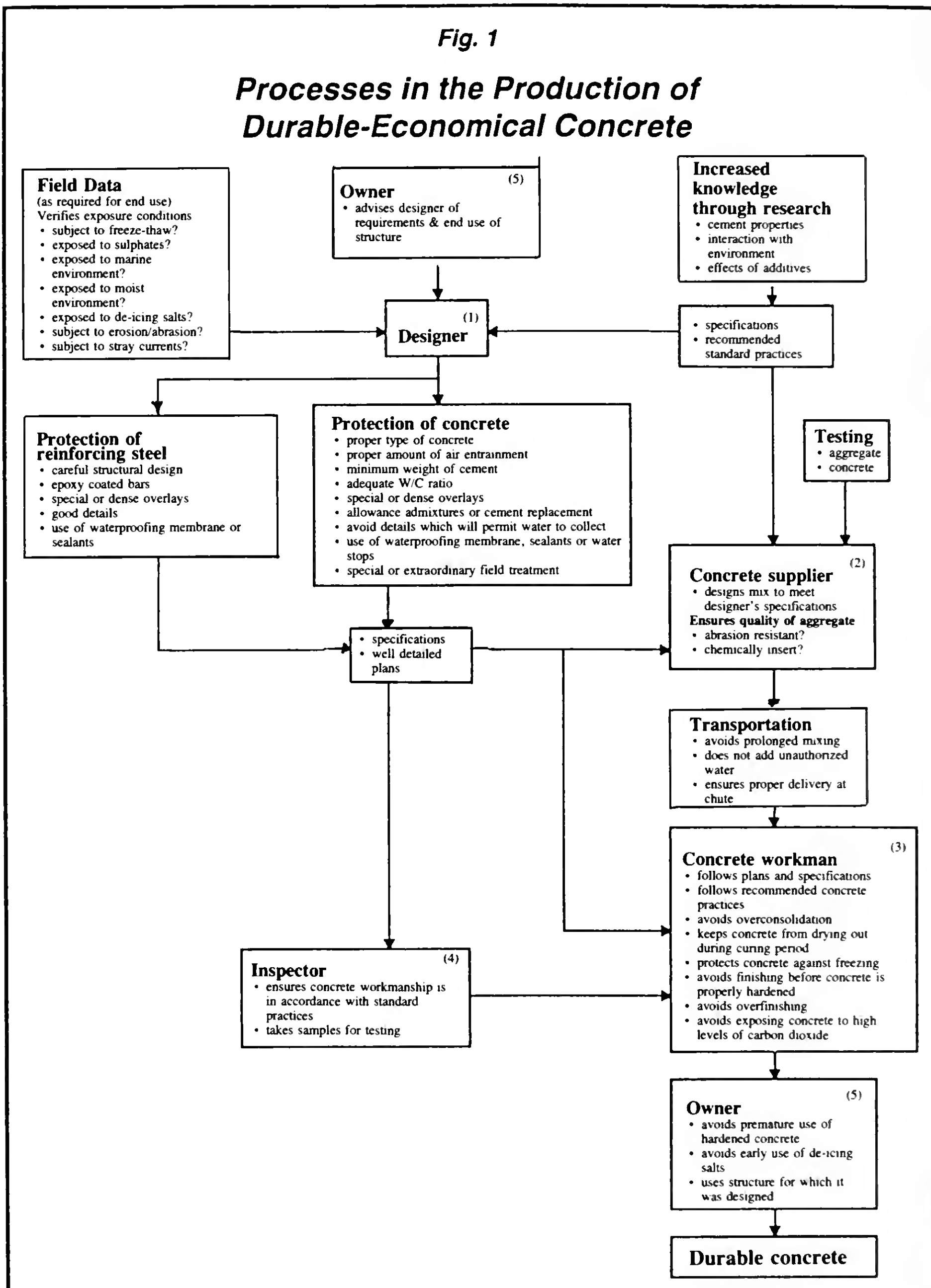

Fig. 1. Processes in the Production of Durable-Economical Concrete

AREA COMMITTEE 16 — ECONOMICS OF PLANT, EQUIPMENT AND OPERATIONS

SURVEY OF RAILROAD APPLICATIONS OF ARTIFICIAL INTELLIGENCE

Chairman: M. W. Franke

Conducted by the Subcommittee Concerning the Application of Artificial Intelligence Techniques Within the Rail Industry

Subcommittee Chairman: C. D. Martland

Abstract

Artificial intelligence techniques have progressed to the point where productive, industrial applications are common. The rail industry has tested or implemented a number of AI applications, primarily involving knowledge-based systems or expert systems. These applications have addressed a wide variety of problems affecting essentially all of the major railroad functions. Continuing development of AI techniques, as well as the rapidly improving technologies for communications and computation, make AI an important potential source of productivity improvements for the rail industry. This paper review selected examples of AI applications in the North American rail industry.

Introduction

Artificial Intelligence (AI) techniques can be defined to include computer programs that exhibit or incorporate reasoning capabilities equivalent to that of humans:

> "AI is the part of computer science concerned with designing intelligent computer systems, that is, systems that exhibit the characteristics we associate with intelligence in human behavior - understanding language, learning, reasoning, solving problems, and so on." [Barr and Feigenbaum, 1981]
>
> "AI is the area of computer science that attempts to provide computers with the capability to interact and react intelligently with their environment and/or to solve problems that would otherwise require human intelligence" [Fink, 1985]

AI techniques certainly include such things as knowledge-based systems, expert systems, neural networks, and natural language systems (see definitions in Exhibit 1). A great many expert systems "shells" are available that provide the inference engine and the basic framework for structuring a knowledge base. Many companies have developed expert systems using such shells. AI techniques are also commonly used in advanced robotics and image-processing systems. AI techniques can be embedded in or integrated with other software applications, notably data base management systems and mathematical models.

AI offers the potential to enhance the capabilities of management at all levels of the organization. Significant advances in computer software and hardware are making AI applications much more readily available, just at a time when downsizing and competitive pressures have increased the demands on managers' time while diminishing technical staffs.

AREA Committee 16 established a subcommittee to examine "Railway Applications of Artificial Intelligence." The Committee had two major goals:

a. Determine the scope of AI activity within the rail industry

b. Disseminate information concerning the most promising applications and the most effective techniques for implementation

To determine the scope of AI activity in the North American rail industry, the Committee conducted a survey of the major railroads. Part I of the survey asked some general questions concerning each railroad's experience with and approach toward AI applications. Part II asked for additional information concerning specific AI applications. It was emphasized that the Committee was concerned with applications that feature AI capabilities, as defined in Exhibit 1. While traditional data base and operations research techniques have proved very beneficial to many railroads, this survey did not address such applications.

Exhibit 1. Terminology

Knowledge-based system (KBS)	A KBS makes domain knowledge explicit and separate from the rest of the system, which also includes general problem-solving knowledge (called an inference engine). The knowledge is often expressed as a series of If-Then rules.
Expert system	A KBS that uses "expert knowledge to attain high levels of performance in a narrow problem area... An expert system must be skillful - apply its knowledge to produce solutions both efficiently and effectively, using the shortcuts or tricks that human experts use to eliminate wasteful or unnecessary calculations." [Waterman, 1985]
Neural networks	A means of storing information patterned upon the processes used in the human brain. Neural networks provide an extremely efficient means of storing patterns and responses.
Natural language system	A computer program that responds to a broad system range of commands expressed in ordinary language so that a user does not need to know detailed technical commands or formats.
Advanced data base interfaces	A computer program that operates in conjunction with a data base and provides assistance to the user in defining problems, accessing data, and producing reports, e.g. a system that structures complex data base queries based upon the user's responses to a series of questions.
Mathematical models with intelligent interfaces	Providing an intelligent interface will make mathematical models much easier to use. For instance, an expert system could be attached to the model to determine what parameters should be used for a particular application.

Survey Responses

Responses were received in 1991 from six railroads, including five major Class I railroads that were developing or had already implemented AI applications. Four of these railroads had formed special groups of 2 to 4 people for AI applications, either within an existing management science department or an information systems department. The earliest group was formed in 1986 and reported six applications. All but one of the five major railroads were increasing their AI activities. Details were provided on 13 specific AI applications.

The reasons for considering AI applications were ranked on a scale of 1 (unimportant) to 5 (very important). The single most important reason was to reduce the routine demands upon the time of experts, freeing them to deal with more pressing systems (Exhibit 2). The next most important motivation was to improve the use of data contained in existing data bases. In some cases, railroads felt that AI offered capabilities beyond what was available in traditional information systems. Capturing knowledge of key employees before they retire was mentioned as a reason for developing expert systems, while several railroads mentioned the potential for using AI techniques in training or in making knowledge accessible to more people.

The most common applications were knowledge-based systems, expert systems, and natural language systems, which were implemented or under development by each of the major railroads. Three of the railroads were involved with advanced data base interfaces, three with neural nets, and one with pattern matching. Many different departments were involved in the AI applications, including marketing and sales, payroll, transportation, rules, locomotive management, car distribution, mechanical, materials, accounting, and information systems.

A variety of computer platforms and software packages were used. All five railroads had developed applications for advanced personal computers, two for advanced workstations, and three for mainframe computers. In all but one application, the railroads used software packages that facilitate development of AI systems. The one exception was a case where the railroad used a "homegrown macro assembler language."

The amount of effort required to develop a workable system varied. A few of the smaller systems were developed in less than 6 calendar months, but most required 6 to 18 months. About half the systems required less than 400 person-hours for programming and implementation; the others required 800 to 1500 hours. Almost all of the reported applications were developed by the recently formed AI development groups; two were developed by other in-house groups working with consultants; and one was previously developed for another railroad.

Exhibit 2. Motivation for Developing AI Applications

	Motivation	Average Ranking
a)	Reduce the amount of time that experts spend dealing with routine matters	4.6
b)	Improve use of existing data bases	4.0
c)	Provide previously unavailable capabilities	
d)	Capture knowledge of key employees in dealing with computers	3.4
e)	Increase general efficiency of all employees in dealing with computers	3.4
f)	Other	
	Standardize applications	5.0
	Spread knowledge across a wider audience	5.0
	Reduce training time	5.0

Sample applications are listed in Exhibit 3. The most common applications addressed the complexities of obtaining accurate routing and pricing information for customer inquiries and billing. Three railroads have developed systems for this area; one reported a 7% increase in the accuracy of billing following implementation. Other systems were designed to handle routine functions, such as screening invoices or making proper updates in payroll deductions when an employee moves.

Respondents were asked to rank the problems encountered on a scale of 1 (no problems) to 5 (severe problems). For the 8 systems that have been implemented, the largest single problem was interfacing the AI applications with the existing information systems (Exhibit 4).

Exhibit 3. Objectives of the AI Applications

Pricing and sales	Allow pricing officers to provide consistent rate quotes
	Generate lists of carriers and junction points for interline moves
Payroll	Edit inputs to payroll accounting system (check tax code and address when employee moves)
Accident investigation	Derailment analysis expert system (to guide data collection and analysis) Ensure 100% compliance with drug testing regulations following accidents

Mechanical	Locomotive wheel management to determine the need for wheel turing and to standardize the decision process
	Locomotive brake and electrical system troubleshooting
Car Distribution	Improve car distribution planning
Service monitoring	Determine the causes of delays when customer targets are not met
Purchasing	Invoice processing (screening for acceptance or rejection)
Transportation	Provide a "Help Desk" for diagnosing computer problems

Exhibit 4. Problems in Developing AI Applications

	Problem	Average Ranking
a)	Developing interfaces with existing information systems	3.5
b)	System design	2.6
d)	Developing user interfaces	2.4
c)	Developing AI capabilities	2.2
f)	User acceptance	
e)	User training	2.0

AAR Activity

In 1986, the Association of American Railroads asked its Affiliated Lab at MIT to assess the potential for AI applications in the rail industry. At that time, the most likely applications were thought to be knowledge-based expert systems (KBES). The MIT Affiliated Lab produced a working paper [Little, 1986] that described KBES, suggested guidelines for choosing a good application, and identified potential railroad applications. One particularly promising application was chosen as a demonstration of the KBES technology. The Car Repair Diagnostic System (RCDS) analyzes freight car maintenance histories, diagnoses certain patterns of excessive usage, and recommends corrective action. In tests using data from two railroads, RCDS found that approximately 1% of the cars in the fleets examined accounted for 3% of the repair costs [Little and Martland, 1990]. RCDS made use of a "structured history" of car maintenance records that has since been adopted by several private fleet owners.

In 1991, the MIT Affiliated Lab produced a broader assessment of potential railroad applications of emerging data technologies, including but not limited to AI applications [Martland, Sheridan, Ben Akiva, Wormley and Sussman, 1991]. One of the most promising applications was the development of decision support techniques that would use artificial intelligence to support operations control. Research was begun in 1992 to demonstrate the use of fuzzy logic in deciding which trains to run and when to run them [Ren and Sheridan, 1992].

General Activity

There have been many expert and knowledge-based systems that have been applied to rail and other transportation problems. In 1986, Transport Canada conducted a thorough survey of expert systems being used in transportation and prepared a primer on the development of expert systems in the transportation sector [Johnson, 1986]. In 1989, an international colloquium was held on *Railway Applications of Knowledge Based and Expert Systems*. The *Proceedings* from this Conference include the following examples of European applications of expert systems:

a. London Underground tested a diagnostic system to perform on-board fault analysis for subway trains [Mulvey and Skingle 1989].

b. Spanish National Railways (RENFE) developed a rule-based system for diagnosing the causes of in-service failures of their most commonly used locomotive [Valdecantos, 1989].

c. German Federal Railway (DB) developed various expert systems for operations safety and quality; one system provides support for the Signalman in the event of a train failure or a deviation from planned service, while another provides support to the person in charge at the scene of an accident [Metz, 1989].

d. Netherlands Railways (NS) has developed several systems, including one that allows the railroad to respond quickly to requests for freight tariffs [Koning, 1989].

e. The French National Railways (SCNF) developed one system for routing trains between passenger platforms and the running tracks in complex stations [Moulin, 1989] and another for empty car distribution [Jean, 1989].

f. The Portuguese Railways (CP) developed a system to assist in crew scheduling, in which the user specifies rules that limit the combinations to be assessed by an optimizing algorithm [Morgado and Martins, 1989].

In addition, two North American applications were presented at this conference; the RCDS prototype discussed above and a rail replacement scheduling prototype developed for Burlington Northern [Martland et al, 1989].

At the Comprail '92 Conference, a paper described how a knowledge-based system is used in designing and evaluating automatic train control systems [Bozzolo et al., 1992]. This system reduces the complexity of various design alternatives in order to facilitate standard performance, reliability, and safety analysis.

Finally, the Council of Logistics Management sponsored a study of the use of applications in logistics (Allen and Helferich, 1990). This study provides a general introduction to expert systems and examines potential applications in specific areas of logistics, including inventory management, order management, transportation, materials handling, and packaging. There is also a chapter on other AI technologies, including intelligent databases.

Discussion and Prognosis

Artificial intelligence techniques have progressed to the point where productive, industrial applications are common, as is evident in the large and growing literature on AI applications. The rail industry, both in North America and abroad, has tested or implemented a number of AI applications, primarily involving knowledge-based systems or expert systems. The AAR has sponsored research to demonstrate the applicability of expert systems and other AI techniques to railroad problems. These applications have addressed a wide variety of problems affecting essentially all of the major railroad functions. Continuing development of AI techniques, as well as the rapidly improving technologies for communications and computation, make AI an important potential source of productivity improvements for the rail industry.

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MEMOIR

Henry W. Keevil
1905-1993

Henry W. Keevil, retired Detector Car Engineer for the Association of American Railroads, died in Evanston, Illinois on January 20, 1993. During a 43-year career with the American Railway Engineering Association and the AAR, Mr. Keevil participated in the development of the first rail flaw detector car and was the principal facilitator and developer of the residual magnetic method of rail flaw detection.

Henry Walter Keevil, the second of three sons of Henry M. and Eva L. Keevil, was born December 14, 1905 in Kansas City, Missouri. His father spent his entire career in the Railway Mail Service, mainly in Kansas City. Henry attended junior college in Kansas City and then transferred to the University of Illinois. He graduated with a degree in Railway Electrical Engineering in 1927.

An expected job with the Chicago Rapid Transit Company failed to materialize; and after a short stint with the Kansas City Public Service Company, Henry joined the detector car development program which was then in progress under the direction of Walter C. Barnes, Engineer of Tests for AREA Committee 4—Rail. The Sperry Development Company was developing the car for the AREA, and Henry joined the development team then headed by Sperry's H. C. Drake at the Sperry laboratory in Brooklyn, New York.

Toward the end of 1928, this first detector car, which used the induction method of flaw detection, was successfully demonstrated on the New York Central line and was accepted by the AREA committee. Henry then took the car on a demonstration tour to railroads all over the country. He continued to operate the car when it went into regular leasing service, while working to improve the reliability and effectiveness of the equipment.

In the mid-1930s, the AAR decided to pursue the possibility of developing a practical system of rail flaw detection using residual magnetism. Henry was brought in to work full time on this development, much of which was done at the Chicago, Burlington & Quincy shops in Aurora, Illinois. By 1939, the residual magnetic detector car had become a practical reality and was in regular testing service.

Although Henry credited Mr. Barnes with the original concept of residual magnetic flaw detection, it was the work of Henry and his colleagues, notably William Brazitis and John Dionne, that turned the idea into reality. While managing the AAR's own detector car leasing service and until his retirement at age 65 in 1970, Henry continued to work on ways to further improve the residual magnetic detection system, and later to adapt it to single-unit rail-highway vehicles.

Although it has now been superseded by the more effective and technologically more sophisticated ultrasonic system, the residual magnetic rail flaw detection system, of which Henry Keevil was the leading developer, played an important role in railroad track maintenance and operating safety, both during the crucial World War II years and for many years thereafter. More than 40 magnetic type detector cars, both on-rail and rail-highway, were built and operated, mostly by AAR member railroads.

In 1942, Henry married Grace Kropf of Chicago. Their two sons, born in 1945 and 1950 respectively, are Walter R., who is Chief Rail Equipment Engineer for the Chicago Transit Authority, and Charles H., a retail sales manager. Henry joined the AREA in 1951 and became a life member in 1977.

Former colleagues remember Henry Keevil as a modest, unassuming but brilliant engineer, tireless and totally dedicated to his work. Even more importantly he was a strong leader who was also thoughtful and compassionate.

Robert C. Faulkner   Walter R. Keevil   Tom Shedd

MEMOIR

Gordon Locklin Patrick Plow
1902-1992

Gordon L. P. Plow, age 90, retired Assistant Chief Engineer of the Canadian National Railways, died August 5, 1992 in Montreal, Quebec.

Mr. Plow was born on June 27, 1902 in St. Albans, Vermont. His parents were John Plow, a native of England, and Hortense Harlow Locklin, of St. Albans. He received a B.Sc. in Civil Engineering from McGill University in 1924.

Mr. Plow began his railroad career as a chainman on the Central Vermont Railway in the summer of 1923. He joined the Engineering Department of the Chicago, Rock Island and Pacific Railroad in 1924, during which time he was posted in Chicago, Kansas, Amarillo, Fritsch (Texas), Cedar Rapids, and Rock Island, and back to Chicago again.

In 1928 he returned to Montreal, where he worked as an Instrumentman for CN. From then until 1936 he worked in a variety of short jobs, including the Bridge Department of the Canadian Pacific Railway in 1930-31, and for Canadian Airways Limited in 1931 doing aerial surveying. In 1936 he rejoined CN as an instrumentman on the Levis and Laurentian Divisions.

In 1941 he joined the Canadian Army as a 2nd Lieutenant. He was promoted to 1st Lieutenant and then Captain in 1942 and served various assignments in Canada. In 1944 he was promoted to Major and served in England with OC 830 (Basic) Detachment until May 1945. From May 1945 until November 1945 he was posted to OC 827 (K) Military Government Detachment, and served time in Belgium, Holland and Germany. He held the France and Germany Star and the Canadian Volunteer Service Medal and Clasp.

In January 1946 he rejoined CN, serving successively as Assistant Division Engineer, Division Engineer, General Maintenance Inspector, and Senior Assistant Engineer. In 1950 he became Engineer of Track for the CN System, and in 1952 he became Assistant Chief Engineer, retiring in June of 1967.

He joined the American Railway Engineering Association in 1948, and was a longtime member of Committee 4—Rail. He became a Life Member of AREA in 1973, and was elected Member Emeritus of Committee 4 shortly thereafter.

Mr. Plow is remembered in his latter years for his work with the Engineering Training Program, under which many young Engineers were brought into Headquarters from line points to be initiated into the mysteries of CN's Headquarters offices. His thoroughness, understanding, literacy and sense of humour are well remembered by those who worked with him on CN and throughout the railroad industry.

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MEMOIR

Earl Leslie Robinson, Jr.

Earl Leslie Robinson, Jr., age 71, retired Assistant Engineer, Atchison, Topeka & Santa Fe Railway, died September 7, 1992 in Pueblo, Colorado. Earl is survived by his wife, Mary, his son Charles and two grandsons.

Mr. Robinson was born on January 9, 1921 in Las Vegas, New Mexico. He began his railroad career with the Santa Fe as an Extra Gang Laborer May 23, 1939 and entered the engineering department as a Chainman in August of 1939. In 1941 he left the railroad to serve with the U.S. Army Corps of Engineers, returning to the Santa Fe in December of 1942. He then served in various positions in the engineering department until March 1, 1959 at which time he was appointed Superintendent at the railroad's Pueblo Slag Ballast Plant. In 1974 he was promoted to Assistant Engineer on the Chief Engineer's Staff. In this position he was responsible for the development and operation of all ballast plants on the Santa Fe.

Mr. Robinson served 4 years in the U.S. Army as an Aircraft Engineering Officer. He attended New Mexico Highlands University, New Mexico School of Mines and New Mexico A & M.

Mr. Robinson became a member of the AREA in 1959 and was active in the association until his death. Earl served on AREA Technical Committees 1, 10 and 34. He also served on the Conference Operating Committee. While on Committees 10 and 34 he served as a Subcommittee Chairman. Earl was especially active on Committee 1 where he served as Chairman of Subcommittees B, 2, and 10. He also served as Vice Chairman of Committees 1 and was Chairman of the Committee from 1973 through 1975. His associates on Committee 1 recently recognized Earl with the status of Member Emeritus.

Mr. Robinson's long and devoted service to the railroad industry and the AREA will be greatly missed.

AREA Committee 1

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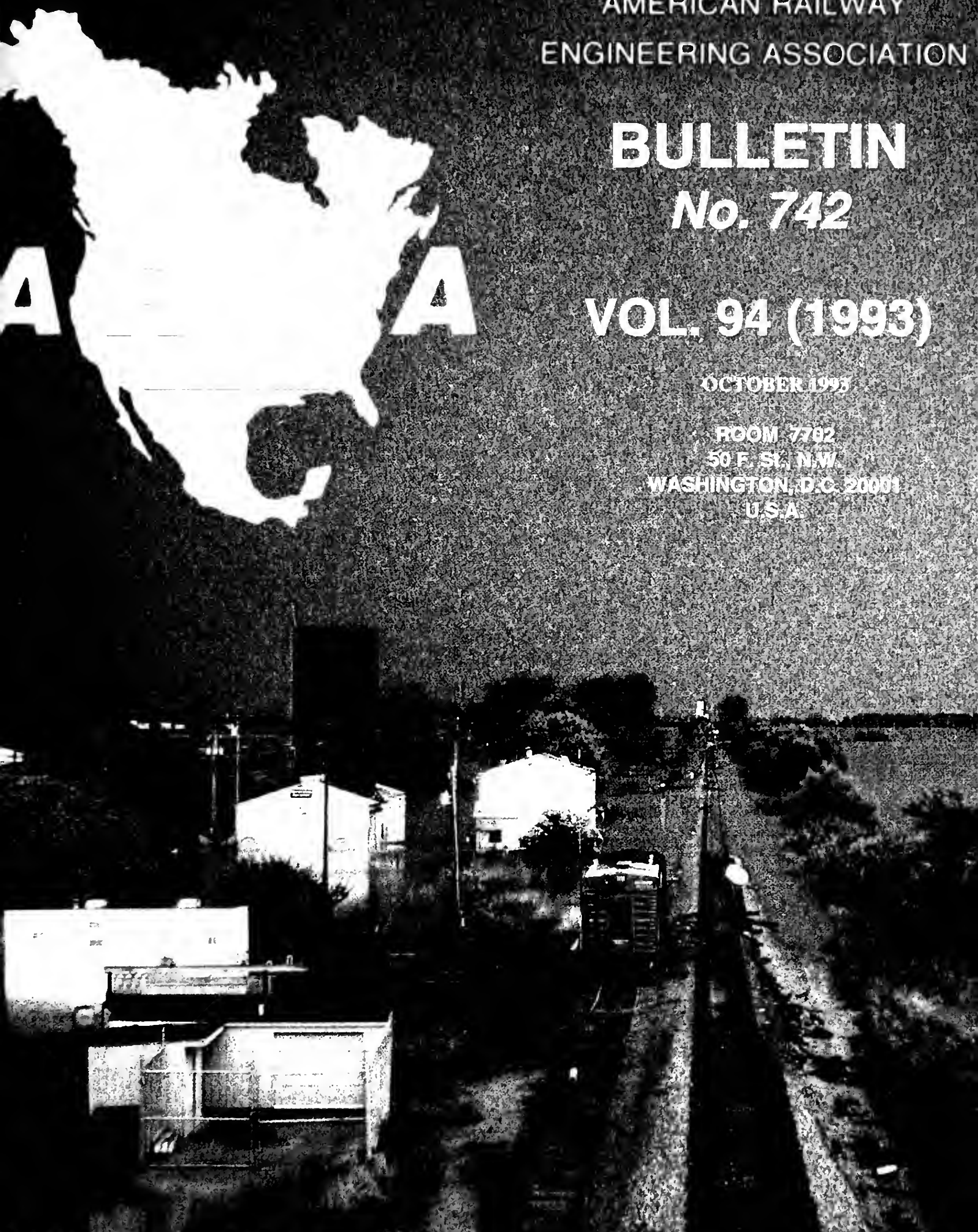

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Cover Photo: Norfolk Southern at Henrietta, Missouri—July 1993

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Rain clouds threaten further flooding as Santa Fe work train with Jordan spreader and ballast cars prepares to leave Henrietta, Missouri July 31, 1993.

RAILROADS vs. 1993 FLOODS

The 1993 floods in Iowa, Nebraska, Kansas, Missouri, and Illinois from June through August have turned out to be one of the greatest natural disaster challenges in railway engineering history. Disruptions to the national railway network through the midwestern U.S. were unprecedented. Only extraordinary efforts kept traffic moving so that shipper and public reaction were generally quite positive about the railroads' response to this challenge.

With the restoration of the UP, Santa Fe and Soo line main lines east of Kansas City the week of August 1, important disruptions of a national nature to the railway network were largely over from the 1993 floods. Although lines at the BN, NS and UP along the Missouri and Mississippi Rivers remained to be put back in service, these had fairly good detour routes, except, of course, to local customers in the flooded areas themselves. Some details regarding the dates of line outages and restorations appear in the September 1993 issue of Railway Track and Structures Magazine.

Probably the greatest damage to roadbed and bridges caused during the midwestern floods of 1993, as opposed to the lines being just flooded, was where lines crossed the Grand River in north central Missouri, and also downstream from the junction of the Grand and Missouri rivers near Brunswick, Missouri. The Grand River had two distinct floods. The first one, which occurred July 10, destroyed a pier on a Soo Line bridge, washed out the Santa Fe's mainline at a small bridge in the Grand River flood plain, and washed out the NS main line west of Brunswick and the Gateway Western west of Glasgow. The second crest, which was higher than the first, occurred on July 25 and 26 after the river completely receded to below flood stage, and undid much of the repairs that had been started from the first flood, plus destroying a pier on the Gateway Western's Missouri River bridge.

This area is important from the national network standpoint, because mainlines of the C&NW, CP Rail (Soo), BN, Santa Fe and NS cross the Grand River within a 60-mile distance. General flooding inundated some lines for long periods with depths up to 15 feet, such as the BN at West Quincy, Missouri.

Each railroad will need to consider whether this was a once-in-a-lifetime occurrence, where putting things back the way they were is considered sufficient; or whether at least some of the lines need to be rebuilt in such a way that they could withstand a similar flood in the future. While none of the Mississippi River bridges were structurally damaged, some had water above the low steel and were subject to high side pressures from the water. It can be said that the lines are now more flood resistant than ever, especially in terms of resistance of roadbeds to scour, due to the dumping of thousands of cars of rip-rap, and in some circumstances tracks were raised permanently and larger waterway openings provided.

It appeared that the main interstate highways through the area of the flood had been designed for a flood that was greater than the railroads had been designed for. It is believed that interstate 70 between St. Louis and Kansas City was never out of service during the flood. Perhaps some legislative or regulatory measure could be taken so that when highways are constructed to higher flood standards than paralleling main line railroads, the negative economic and environmental effect of the modal shift in future floods could be taken into consideration in such a way to justify government expense for increasing the flood resistance of the railroad through raising the tracks or adding additional bridges for flood flow.

Factors which have changed from the past to make this type of flood more challenging to the railroads include a less dense network of detour routes (one of the more significant detour routes used in this flood was an abandoned but was still intact NS line, part of which had been sold to a short line operator. This was from Albia, Iowa to Moberly, Missouri via La Plata, Missouri). Because of the heavier axle loads and specialized equipment, such as the double stacks, even when detour lines exist they may have not have the clearances and structural strength for the detour trains. Techniques for crossing flooded but still intact portions of lines, such as pushing a long train in from one side of the flooded section with locomotives and then pulling it out from the other end with another set of locomotives, are not as useful as they used to be in the past. This is because in the past the vast majority of cars could be run through water 18 inches or 2 feet above the rail without damaging the cargo, whereas

Soo Line Bridge over DesMoines River at Ottumwa, Iowa, July 1993.

Work in progress to restore and lengthen washed out bridge on Santa Fe west of Mendon, Missouri between first and second flood on Grand River.

now double stack cargo could be damaged as low as 9 inches above the top of rail and many hoppers also extend closer to the top of rail than previously.

Another area to consider is levees. Many levees had the effect of stopping railroad operation, and it appears that the changes in importance of the continuity of operation of individual railroad lines to the nation mentioned in the previous paragraph may need to be more fully taken into account by public agencies in future decisions.

Some railroads may wish to consider joint railroad agreements for retaining lines and connections simply as detour routes, as the cost of maintaining a line without trains can be quite low, especially if the condition of the line is good when it would have been abandoned. The detour routes can be used in many other emergencies in addition to floods.

Despite all the difficulties mentioned, the railroads were able to keep the national rail network functioning. The railroad industry and the nation as a whole owe a great deal of gratitude to those many railroaders who worked long, hard, frustrating, and often heartbreaking hours day after day, losing count of whether it was weekend or weekday. This muddy and soaked environment continued for well over a month for many individuals. In the end, it was a triumph of the spirit that keeps railroading the modern, forward-looking industry that it is, a triumph of individuals dedicated to their profession.

CANADIAN EXPERIENCE WITH FRA TRACK SAFETY STANDARDS

By: M. D. Roney*

Introduction

In 1992, Transport Canada, the federal agency regulating Canadian railways, decreed that Canada would have a set of government track safety standards. The railway view that they were ready for self-regulated performance based standards was not shared by Transport Canada. Transport Canada preferred customized FRA standards. This was opposed by the Canadian railways who felt that any departure from well-tried FRA standards could have serious consequences for train operations. So, in September 1992, Canadian railways became legally subject to FRA Track Safety Standards with minor additions in the area of rail wear limits, anchoring and track buckling.

In fact, regulation came at the same time that CN and CP were integrating substantial U.S. mileage into their networks, and had already departed on the long road towards standardization. So the concept of uniformly applying Uncle Sam's standards across North America was not out of sync with their objectives. CP Rail System merely applied tighter thresholds to these same defects to account for the fact that it has always been our policy to work well within "safety" limits.

But when faced with actually working with FRA Track Safety Standards for the first time on a large scale, it was discovered that in spite of a good set of CP geometry standards, operations were not always within FRA's concept of safety. And in fact the lessons of a 1990's railway adapting to 1970's standards may well be of some interest to American railway as they revisit the question of how to modernize the FRA Track Safety Standards.

FRA Standards vs. CP Geometry Standards

The most fundamental difference is that the CP track classes were tonnage-based, while the FRA track classes are speed-based.

On CP Rail System, tightening geometry limits with increasing tonnage was consistent with the track standards policy. Derailment experience has shown that risk is more related to the number of trains than the speed at which they are travelling. So, for example, more than 60% of track-caused derailments on CP in the last 10 years have been in track with tonnages exceeding 10MGT. Only 20% have been at speeds exceeding 40 mph. As a general statement, strict use of speed-based standards like those based upon FRA underprotects slower speed, high tonnage track, such as coal lines. On the other hand, tonnage-based standards underprotect higher speed, low tonnage lines, such as lines dominated by intercity passenger traffic. As it is the nature of CP that the great majority of ton-miles are concentrated on higher tonnage FRA Class 2-4 lines, it is no coincidence that tonnage-based standards had served to keep them on the track in the past. With the exception of truck hunting, the severe dynamic interactions which cause derailments are related to resonance conditions which can occur at a whole range of different speeds anyway.

The second fundamental difference between CP and FRA geometry standards relates to the measurement baseline or chord lengths used.

FRA standards are mostly related to 31 or 62 ft. baselines. This appears to be done for convenience of hand measurement as opposed to any particular significance of this measurement to trains.

In adding FRA defects to the standards, the view was taken that track speed would be maintained one speed class higher than the safety standard similar to the approach of most Class I U.S. railways. This is a good approach but it does not necessarily ensure trouble-free railroading.

On the one hand, even maintenance of track to one speed class higher than required by FRA Standards allows gauge that is too wide for effective operation of low speed, high tonnage lines.

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*Manager Track Technology, CP Rail System

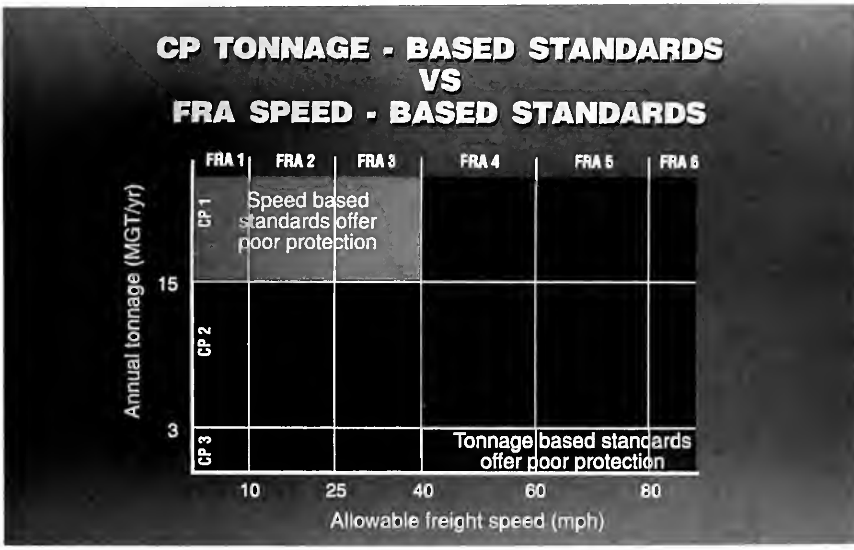

Figure 1 - Tonnage vs Speed Standards

On the other hand, the 62' baseline used in FRA measurements is considerably longer than truck centre spacings seen on the vast majority of our freight car fleet.

It is not a bad baseline to use in a high speed track where longer wavelength track errors can set up a vehicle resonance, but they do not have much high speed track. In fact, unless traffic is completely uniform, it is not a good idea to restrict your measurement baseline to any particular wavelength.

Bruce Bradshaw of Pandrol Jackson uses the analogy of listening to a symphony. The short wavelengths could be the flute and the longer wavelengths the base run. Everyone agrees that we can hear the clash of the symbols, but some of us may key on the role of different instruments in the piece.

When tamping track, all wavelengths are repaired to some degree but the effectiveness drops off rapidly at the two ends of this range, particularly with the older tampers which start to lose some effectiveness in repairing track errors with a span of greater than 60 ft.

One problem with looking for track errors with a 62 ft. chord is that this is at the far end of what a tamper can smooth out of track, for precisely the reason that these long chords are of little concern to any but the longest cars and fastest trains.

Therefore, the AAR proposal to change the FRA Standard for deviation from uniform profile to look at all chords up to 62' is supported.

Ironically, while the 62 ft. chord is too long a chord for typical freight speeds, the 31 ft. chord used in spirals is probably too short. This is because wheel unloading in spirals is more related to the constraints of tight geometry than to a speed-related dynamic interaction.

For example, Figure 3 compares two measurements of track twist in a spiral. FRA specifies a 31' baseline. CP uses both a 55 ft. and a 20 ft. twist baseline. The 55 ft. baseline is of particular significance. CP Rail System has experienced too many derailments in the past when there is more than 2 in. change in elevation between diagonally opposite corners of empty 112J series tank cars. 55 ft. corresponds to the wheelbase of these very torsionally rigid cars and is directly related to the tendency for wheel lift at any speed.

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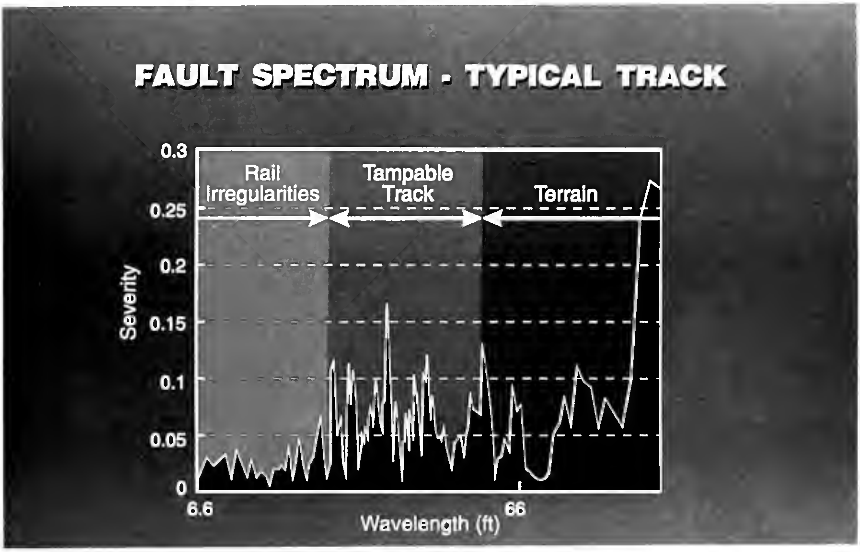

Figure 2 - Fault Spectrum - Typical Track

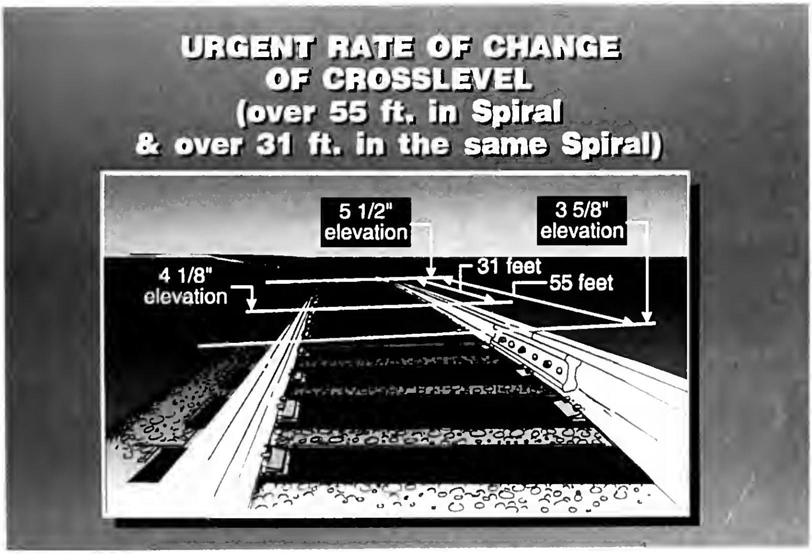

Figure 3 - 31 ft. vs 55 ft. Twist in a Curve Spiral

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CPRS also uses a 20' twist defect, which corresponds to the half length of a 39 ft. rail to specifically find low joints. Low joints are often the primary input to a range of different geometry problems. A short 22' chord is used to identify locations where a single axle would respond to a sudden impact.

It would appear that the 31' Spiral Crosslevel measurement specified in FRA Standards is a compromise between these critical wavelengths, which serves neither purpose very well.

But in addition to the wavelength of different errors in track, what is really important in preventing derailments is probably the periodic sequence of track errors that individually comply with FRA but are spaced at just the right distance apart to set up freight car oscillation. This fact is considered in CP's geometry standards, which incorporate two "rock and roll" geometry defects that look for characteristic repeating track signatures. U.S. railways who use the TSC's Crosslevel Index are doing the same thing. To return to the music analogy, any railway that strictly applies FRA's single location defects can be sure that "ROCK AND ROLL IS HERE TO STAY".

Another type of track condition that CPRS has felt figured in more than one derailment was a rapid change in gauge. Some of these locations would be a wide gauge or a narrow gauge condition, or perhaps an alignment error, but not all. The direct cause is the fact that as one rail moves inward, it forces the axle over towards the opposite rail, where tight gauge may mean that rail is already in contact with the throat of the wheel.

Neither of these three situations is well covered by existing FRA Track Safety Standards, so existing geometry defects were retained to cover them.

How Government Standards Have Helped

So CP had independently applied a good set of track geometry standards in the past, implementation last year of FRA Standards nevertheless taught CP some things about track. Twenty years of heavy reliance on mechanized maintenance had left the track smooth, but different from its original design.

In relying on tampers to smooth the track, we lost the long range view of the experienced track foreman were lost, whose eye may not have accepted the long 62 ft. sags in our jointed track.

But the greatest grief came from what FRA Standards said about Class 4 and 5 curves. CP's geometry car had always rated the design speed of curves based upon average measured curvature and superelevation. But never was it specifically diagnosed whether there was a match of superelevation through every foot of a curve.

While the curves undoubtedly started with a good design geometry, repeated smoothing by tampers had left FRA violations in some territories. The culprits were the 3 or 4 pt. tamper reference systems and inexperienced operators who did not appear to fully appreciate that the trace made by the tamper is not a true reflection of the curve geometry.

Starting from your right and moving left, the tamper "sees" the curve before the workhead is on the start of the curve. Unless the operator knows that there is a fixed offset related to the ratio of his measurement chords, or is using a newer generation tamper that automatically factors this in, curvature may be started too early. The result can be an FRA violation due to unbalanced elevation. The opposite can be seen at the other end of the curve, where versines can be reduced prematurely, leading to overelevation and more FRA violations. If the tamper is run in the opposite direction the next time, the curve is shifted back in the other direction, but by this time trains will have compounded the error. In fact the LASERAIL rail wear measurements give a pretty good picture of what trains ultimately thought about the way individual curves were lined.

Furthermore, if the tamper/liner operator is not specifically working to predetermined TS and SC locations, there is a tendency to bunch the steel up at both locations, again leading to unbalanced elevations.

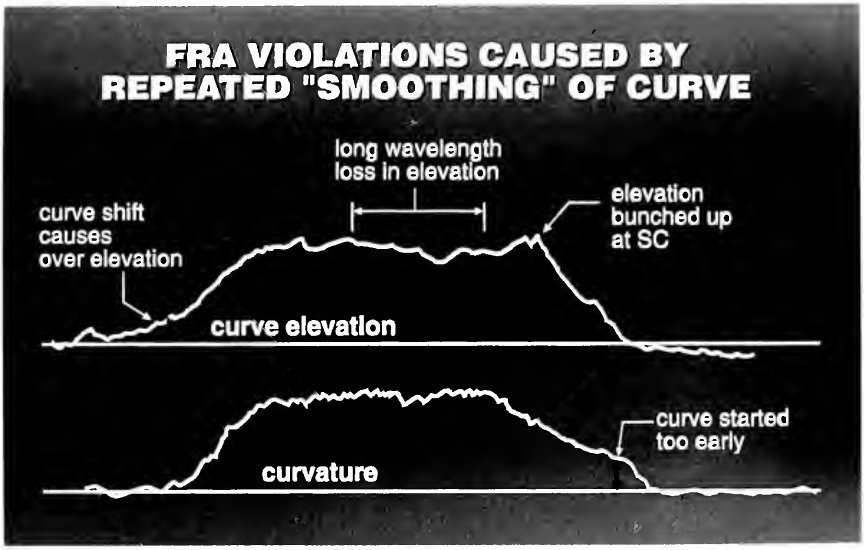

Figure 4 - FRA Violations Caused by Repeated "Smoothing" of a Curve
How Government Standards Have Helped

In the curve body, the same 3 or 4 pt. measurement system that is too long to use in the spirals without correction factors, is too short to see longer wavelength loss in curve elevation. Again, intelligent use of tampers, or use of the newer machines will prevent underbalanced superelevation errors.

So to summarize, CP Rail System has found that it is necessary to measure the same track geometry parameter using several different chords or baselines. These should relate to truck centre spacings of critical equipment or to wavelengths that are known to cause a strong equipment response. This has been done effectively at CPRS by merging FRA defect definitions with existing CP standards. The railways need "rock and roll" defects in addition to individual defect standards, and should operate considerably tighter than required by FRA in slow speed high tonnage track.

The Impact of the Change

Figure 6 shows the number of urgent defects per 100 track miles that were found by CP's geometry car at each of 4 runs over randomly selected mainlines in Eastern Canada vs the former Soo Line.

An "Urgent" defect is a mandatory slow order, and includes all CP safety and FRA violations. Of the 4 tests performed each year, the summer and fall runs were chosen as they are least susceptible to seasonal factors. The centre of the diagram marks the date of the changes in standards. Prior to summer '92, CPRS used exclusively CP internal track standards. Starting in summer '92, FRA defects were programmed onto track geometry cars. As previously discussed, defects which uniquely protected CP from derailments, were kept.

The results show a complete reversal in the fortunes of these two territories. In 1990, CP had subjected the former Soo Line to CP track geometry standards. The aftermath, was still a very high rate of about 150 urgent defects per 100 miles. Track evaluation consist was subsequently dubbed the "train from hell" by Soo Line Roadmasters. Most of these defects were related to low joints and to CP's tighter

standards for gauge. But by the time many urgent CP defects were replaced with FRA Standards, the Soo people had been very effective at getting these defects out of track for good, and this gratifying trend has continued into the present.

The reverse happened on our Intermodal Freight Systems Business Unit in Eastern Canada. Used to very few urgent defects, it was tough on proud Roadmasters when high curvatures and higher speed lines suddenly became major defect generators. It is fortunate that FRA defects were chosen to be implemented four months earlier while work gangs were still out on track for redeployment.

In Eastern Canada, the problems were FRA's long 62' twist defect in tangent jointed track and unbalanced superelevation in our Class 4 and 5 track. The curve violations were the result of the sharp curves typical of Canadian shield country along with their practice of running with 2 in. unbalanced elevation. This left little margin for sloppy curve lining. The lesson is that if you have never specifically looked for track problems as defined by the FRA definitions, you will never know if they are there.

But, just as happened on the Soo Line when CP geometry defects were implemented, now that they have been made aware of what part of the symphony we are listening to, they have been effective at getting most of these locations repaired.

Unfortunately, the music lesson has not been without consequences for the smooth operation of trains.

Although the number of slow orders has been declining, the effect on each train in this 450 mi. corridor changed from an average of a 35 minute increase in transit time, to a high of 90 minutes after implementation of FRA Standards. Clearly Transport Canada was not popular with our General Managers.

But Transport Canada and their standards were popular with a handful of Roadmasters in jointed territories, who had felt that speeds may have been too high for the type of track they were being asked to maintain. As a result of implementation of FRA defects, four subdivisions have had their timetable speeds permanently reduced. Freight speeds have been capped at 60 mph for the time being, and some passenger train speeds were restricted until the frost is out of the ground.

On closer examination, fully 65% of our FRA violations have been due to borderline measurements. It is either 30 mph track that just barely meets the FRA requirement for Class 3 - 40 mph operation, or 50 mph track that just barely meets the FRA requirement for Class 4, or 60 mph operation. The majority of slow orders would be eliminated if the FRA defect threshold were interpolated to 30 mph for Class 2, 50 mph for Class 3 and 65 mph for Class 4. The arbitrary definition of speeds defining each FRA Class have a big effect on operations, as there was never had the opportunity, back in the last century, to design our curves for the 25, 40 or 60 mph freight operation which fits well with FRA Track Safety Standards.

Also, the obvious tendency to slow order track with a Class 3 violation to the maximum allowed for Class 2, can run afoul of freight car resonance. The maximum speed of 25 mph for Class 3 is just too close to the 15-25 mph range where dormant, rock and roll conditions can rear their ugly heads.

But a positive result of conversion to FRA standards has been an improved consistency in our setting of slow orders. When CPRS defects were tonnage-based, the Roadmaster's instructions were merely to protect immediately with a slow order. 10 and 20 mph slow orders were common. In many cases, these forced trains into large speed reductions and subsequent accelerations which could themselves cause large buff loads. Excessive slow orders also result in overloading of low rails.

Now with Transport Canada/FRA defects, the great majority of our slow orders take us down only to the maximum permissible speed for the next less restrictive FRA class.


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Is CP Rail System a Safer Railway Now?

CP Rail System was already at or near the top in numbers of FRA-reporting derailments per million train miles due to track and structure, even before FRA Track Safety Standards. But there has been, in fact, improvement.

The Canadian portion of CP Rail System has historically experienced half of the derailment rate of U.S. Class I railways. Through 1992 to November, which includes implementation of FRA standards, track-caused derailment rate was reduced by 25%, but U.S. Class I's reduced their derailment rate by 30%. So, are they safer because of new track geometry standards, or, like most major railways, is it simply because there exists concentrated tonnage on upgraded mainline track and they are smarter about how to inspect and maintain?

More than safety standards, CP's improved safety can be partly related to the fact that track geometry data now routinely finds its way into track maintenance planning systems which recommend programmed maintenance priorities.

To investigate this question of safety further, recently the AAR's FEEST boxcar was run with track geometry consist.

The FEEST car is designed to drop a paint mark whenever the lateral or vertical loadings imposed on a boxcar exceed the fatigue limits assumed in freight car design.

CPRS ran the FEEST car in the geometry consist between Minneapolis and Chicago in the fall of 1992. Over this routing, the AAR car recorded 24 locations where stresses were imposed on the white FEEST boxcar that were at or near to the design loading for freight car fatigue.

In 6 of these cases, no apparent cause of the high loadings on the boxcar was found. At two locations there were FRA violations, and at three different locations, violations of CP geometry defect standards which were retained were recorded. Of great interest is the fact that at 6 locations, there was a substantial change in track modulus. One was a tunnel approach, one a bridge approach. Another was at a swampy area, and another was at a location disturbed by track maintenance.

As expected, 3 of the high impacts were at turnouts and 6 more were at crossings, traditional maintenance problem locations which also characteristically represent a change in track modulus.

Vehicle/track resonance appeared to be a factor, as 8 locations showed patterns of periodic variation in surface and crosslevel while 3 more had periodic alignment irregularities. Finally, 2 locations showed a combination of surface and alignment errors, with neither being of sufficient amplitude to individually trigger a track geometry defect.

There is far more to track safety than can be seen from individual track geometry defects. One particularly puzzling piece of evidence is that there were an equal number of similarly "bad" sites that did not excite the FEEST boxcar to dump paint. The results encouraged getting prototype track modulus measurement on the geometry car into a production role. And it pointed to the future of track inspection as being a performance-based measurement of the dynamic behaviour of critical freight car types.

The obvious final question is - Could some of the mishaps of the past been prevented if FRA had implemented Track Safety Standards earlier? To answer this, the track measurements were dug out at the sites of 50 derailments between 1981 and 1992 which had been attributed at least in part to a track geometry problem. It was then reviewed whether, if there had been a geometry car over these sites the day before the derailment, then the condition was repaired and slow ordered, hence changing the shape of history by preventing the mishap.

This was looked at presuming that the geometry car had been programmed with the former CP track geometry standards, and again assuming only FRA Track Safety Standards.

The results showed that 20 of these derailments would have occurred anyway, whether or not they had tested for geometry, for there were no violations of either CP or FRA Standards. In 16 of these 50

cases, both CP and the FRA would have agreed that this was not acceptable track, and either standards would have prevented the derailment. In 12 cases, the CP standards would have slow ordered the track, while there would not have been any FRA violation. Again, the usual scenario was a long wavelength twist in a spiral, a rock-and-roll problem, or a 1 inch wide gauge condition in a low tonnage track. Finally, 2 of these incidents might have been prevented if FRA Standards had been applied. Ironically, both of these related to overelevation in poorly maintained curves, which apparently contributed to wheel climb derailments.

Conclusion

In conclusion, the experience of CP Rail System with FRA Track Safety Standards strongly indicates that track safety standards should evolve continuously. Track defects are rarely a black and white issue. Any railway using FRA Track Safety Standards, even if maintained to one speed class higher, should supplement these with some geometry defects based upon the specific wavelengths that cause dynamic problems for their most critical car types. This would be particularly true of railways carrying heavy tonnages on lines maintained only to meet FRA Class 2 or 3 standards. Particular attention should be paid to turnout and crossing locations, and any other location where track stiffness may change abruptly.

With modern track measurements, it is no longer necessary to use 31 ft. and 62 ft. measurement baselines. It would be logical to extend or reduce these baselines with the FRA Track Class as the critical wavelength will in fact extend with speed. Because train operations can be critically affected by threshold violations, there should be more flexibility in the speed classes. Linear interpolation of the slow order speed commensurate with the size of the track error would be one solution.

In the end, track geometry standards are merely a proxy for bad vehicle/track interaction. Some 80-90% of derailments are the result of coincident failures in the vehicle/track/human system.

Future improvements in our ability to anticipate and correct potential track problems will undoubtedly come about through three developments:

1. Implementation of regular direct measurement of freight car behaviour and track response to this loading;

2. Development of improved technologies to directly assess tie condition;

3. Continued implementation of computer-assisted track maintenance planning systems which direct maintenance effort in advance to the roughest track locations before problems can develop.

Track safety is truly a Quality Process. It can be improved by regular and direct feed of track information in the form of projected Track Quality Indices through the work planning process, which is exactly the way CP Rail System and many other Class I's have been headed anyway.

DOUBLING SPAN LENGTH ON AN IN-PLACE BRIDGE DURING FLOOD CONDITIONS

By: Kenneth L. Wammel, PE* and Donald F. Sorgenfrei, PE**

The Southern Pacific's main line between Houston and New Orleans crosses the Trinity River near Liberty, Texas. This Line carries approximately 25 MGT annually consisting of mixed freight and 2 daily Amtrak trains. Flood conditions on the Trinity River in May 1990 and January 1992 were reported extensively by national news media. Unique investigation and construction techniques utilized during both floods are the subject of this presentation, and include: the determination of river bottom conditions during flood conditions, the placement of tremie concrete under a pier during a flood, and doubling the length of a span when the pivot pier was lost beneath a swing span while in the closed position.

The Trinity River is 550 miles in length with a drainage area of 17,969 square miles. The head waters are north of the Dallas-Ft. Worth area and terminates at Trinity Bay/Galveston Bay, approximately 41 miles downstream from the railroad bridge. Heavy rains can occur at any time of the year and generally high water will last several months. Approximately 75 miles upstream of the bridge is the Livingston Lake/Dam, a lake for recreational use and water supply. During flood conditions the dam provides little flood control and must release most of the flood flow as it receives it since the dam does not have a spillway and cannot allow overtopping. The Trinity River at the bridge site has a normal flow of about 7,000 cfs and a flood flow of 20,000 cfs. In May 1990 after a long period of rain in the Dallas area, the river's discharge at the crossing swelled to 100,800 cfs at a flow rate of about 6 fps with the water level over the pier tops and within 7 inches of the bridge low steel. A flood of similar proportions occurred in January 1992.

The Trinity River Bridge (Figure No.1) was built in 1896 with a crossing length of about 1530 ft. consisting of a 225 ft long swing span and a single 225 ft long thru truss span flanked by open deck timber spans. During flood conditions, the river overflows the banks and is partially carried through the approach structures. The flood flow additionally is carried through 9 timber structures totaling some 3,016 ft in length over the 3 mile wide flood plain. The river/bridge site configuration is less than ideal with the river being "S" shaped at the bridge crossing and flood waters strike the piers at about a 45 degree angle. The original simple double-intersecting span was replaced in 1904 with a 225 ft long pin connected Pratt truss and the swing span was replaced in 1919 with a 2-span continuous type swing span. There were no requirements for operating the swing span in recent years and the span officially became a "fixed" span. The substructure for the truss spans are stone shafts atop timber mats and supported by timber piles. The piles were estimated to be 55 ft long. In the 1950's, following a reported earlier scour condition, the pivot pier and east rest pier were encased by steel sheetpile cofferdams, filled with sand and gravel and sealed with a 3 ft thick non-reinforced concrete cap.

In mid-May 1990 the river rose and remained at a sustained high level. On May 18th survey controls were placed on the superstructure at the east rest pier, Pier 2, to monitor for pier settlement and tipping since by this time the pier was completely submerged in the flood waters. Pier movement occurred gradually over the next few days with a loss in elevation on the downstream side and a corresponding horizontal movement. By May 23rd the movement reached 5 inches downward at the downstream pier end and 4.5 inches horizontal movement. At this time train operations were halted and detour routes established. Without live load on the bridge, no additional pier settlement occurred.

Sounding the river bottom using conventional lead line means was not practical and it was unsafe to try and place a boat in the flood waters. A depth finder used from the bridge deck yielded only a few isolated depth readings and air bubbles from water turbulence interfered with fathometer readings. Since

it was believed that the settlement was due to scour at the downstream pier base, an accurate river contour was imperative for developing repair plans. The prospect of having to drill a hole down through the pier to investigate for undermining seemed to be a drastic measure. It would be used only if all other exploration methods failed.

*Southern Pacific Transportation Company

**Modjeski And Masters, Inc.

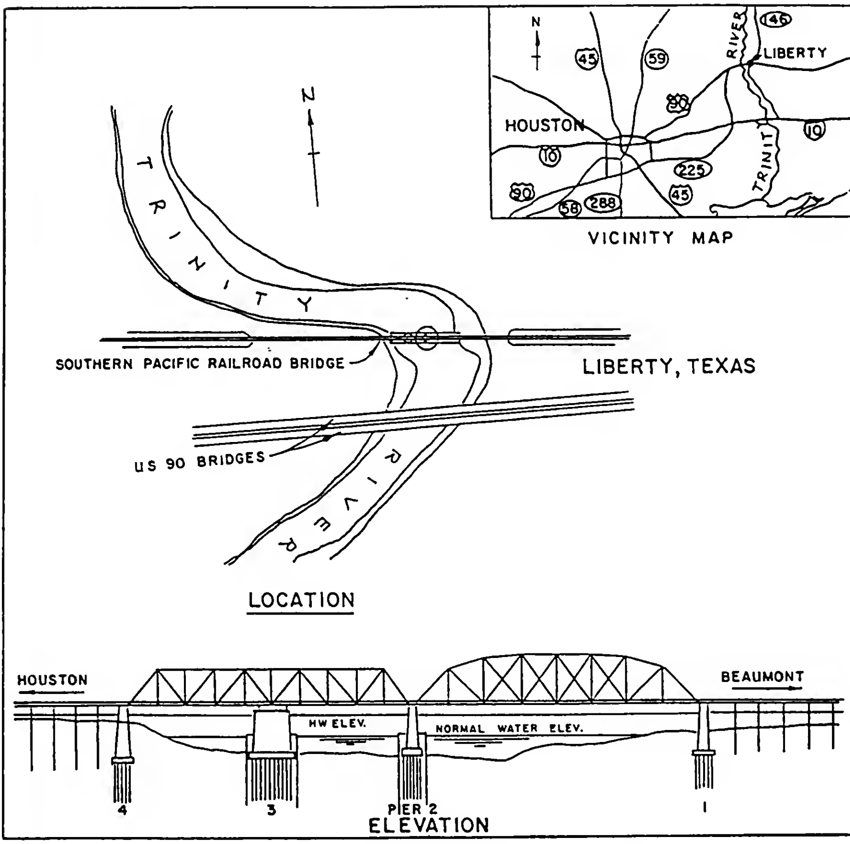

Figure No. 1 – Trinity River Bridge, Liberty, Texas

The use of a profile sonar was discussed with the Wimple Group of Houston. Their primary application for sonars was in the offshore oil field, checking the ocean bottom coverage over pipelines. After a few hours of swapping technologies and persistence in experimenting with their equipment and once there was an understanding of the various orientation options such as polar coordinates, horizontal imaging and vertical profiling, the sonar images were readily understood. The sonar images confirmed extreme scour conditions existed off the southwest corner of the pier with a trench of about 30 or more feet deeper than the normal river bottom and as close as 20 ft from the pier cofferdam. However, the most important discovery was the finding that the cofferdam concrete cap had failed in the southwest corner and the sand/gravel backfill had been washed out of the interior. The sonar image (Figure No.2) showed the base of the pier timber mat and the undermined pier corner exposing approximately 12 ft of pile length. The loss of skin friction on the downstream piles apparently caused the settlement. The sonar was oriented in various directions sufficient to determine that the cofferdam sheet piles were intact.

Given the fact that the sonar showed excessive pile exposure on the downstream side of the pier and the survey data confirmed that the pier had rotated, several options were explored for stabilizing the pier. All options involving the driving of piling in the river to secure the spans were quickly ruled out because the rail mounted piledriver could not hold piles in the leads with the calculated water pressure.

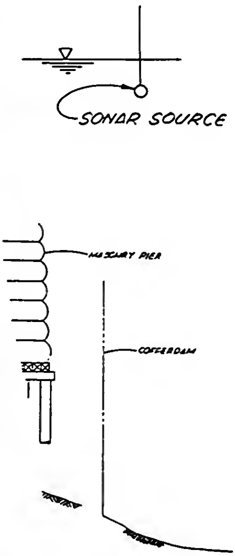

Figure No. 2 – Sonar Image (Left); Interpretation (Right)

There were no marine options, since there were no barges and tugs upstream of the bridge and the river level prevented bringing such equipment from the downstream side due to the adjacent highway bridge. It was believed that we could fill the void with tremie concrete. With a good portion of the original cofferdam concrete cap providing some protection from the swift flow and placing the tremie pump pipe angled under the pier, there would be a good chance to restore sufficient skin friction on the exposed piles or at least the tremie would fill the void under the pier with a low grade concrete sufficient to have some bearing value until the flood condition passed.

A 7 sack tremie concrete was placed using a local batch plant to supply concrete to the site. The concrete was pumped from the ground level to concrete trucks secured on flat cars which were rail transported to the site, dumped into a second pump connected to a stationary rigid pipe angled within the cofferdam for maximum reach under the pier. During this tremie process, the sonar was used to monitor the void under the pier to see if the tremie was effective or being washed out. The sonar was able to pick-up the mounding of the tremie concrete beneath the pier. The concrete pour was stopped at 224 yds which filled the downstream void to what appeared on the sonar monitor to be about 3-4 ft short of contact with the bottom of the pier timber mat. There was concern that the amount of concrete placed against the cofferdam sheet piles might cause a blowout since the cofferdam only had several feet of embedment at the southwest corner and there were no wales or a tieback system for supporting the sheetpiles. It was a needless risk to continue concrete placement. If additional concrete was needed, it could be placed several days later. It was also decided that by leaving a gap under the pier mat, the pier could be underpinned when the water receded. After letting the tremie concrete cure for 3 days it was determined by compressive strength test cylinder results that sufficient concrete strength had been achieved and traffic could be restored. Trains crossed at a slow order speed of 10 mph and a survey party kept a constant monitor on the pier for a period of time. A very minimal initial settlement occurred and thereafter none were observed. The river gradually receded and first exposed a tilted pier top with a 7.75 inch lean across the top and later the cofferdam sheet pile top was exposed, somewhat out of shape but intact.

Permanent pier repairs by underpinning methods (Figure No.3) proved feasible when the divers confirmed that sufficient gap remained between the pier base and the top of the recently placed tremie concrete for placement of underpinning beams. At the upstream end of the pier there was very little tremie concrete and mainly river bottom material which could easily be removed by air lift methods. Because of the known depth of potential scour during flood conditions, it was decided to drive four clusters of 6 H-piles each to refusal some 60 ft into the river bed, cap the piles with a grillage which would distribute the beam load to the piles while assisting in providing a surface for sliding in the underpinning beams through a window in the side of the cofferdam. Reinforcing steel cages would be set around the pier within the cofferdam and the entire cofferdam would be tremie sealed to above the base course of the pier shaft.

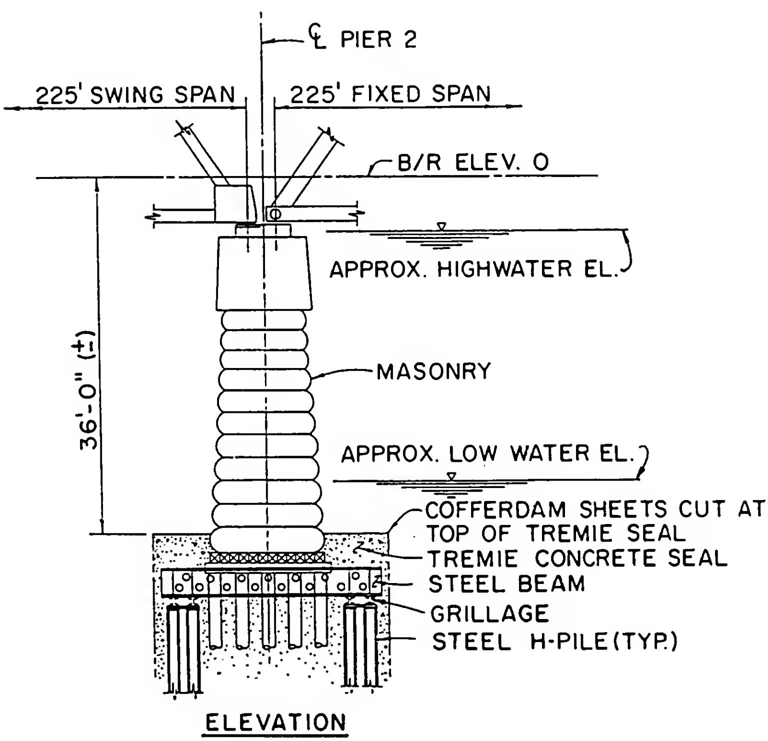

Figure No. 3 – Underpinned Pier 2

Initially the SP intended to contract the underpinning work but there were many unknowns in the process, therefore the Railroad elected to do the work in-house and subcontract certain items. A field office and cook trailer were setup at the site for the convenience of the B & B workers and various subcontractor employees.

The remaining portions of the concrete cap were removed with great difficulty. Much of the 3 ft thick cap had broken into large pieces during the flood and jammed in an irregular pattern. Bristar was used to break some of the pieces into manageable sizes and then a diamond wire cutter was used to cut-up the remaining pieces. There were pieces of concrete, an old timber cofferdam and debris which interfered with driving piles. To expedite pile driving, holes were cored through the downstream tremie concrete for driving the piles within the cofferdam. The underpinning process used an extensive amount of diver time to air lift material to clear the zone for the underpinning beams, cut the piles to grade, set the grillages and beams, and after tremie sealing the pier, cut the cofferdam to the top of the seal. There was concern that the tremie concrete would be disturbed during placement by the presence of the underpinning beams. It was feared that the concrete might spill over from one compartment between beams to the next compartment, thereby washing the cement out of the concrete. To avoid this potential problem, a three pipe-to-three pump assemblies setup was employed with all three assemblies in simultaneous use. Three pipes were set to the west side for the concrete to flow eastward between the sets of underpinning beams until the concrete reached the top of the beams. Two sets of 3 pipes with an "L" shape for reaching under the pier to the pier center were set at the level of the underpinning beam tops from the east side. One set of pipes aligned with the downstream beams and one set aligned with the upstream set of beams. These latter 2 sets of pipes would allow pumping concrete under the pier mat until it flowed outward and built upward to the first stone course. This technique would assure no air pockets under the pier. Soundings were frequently taken atop the placed tremie concrete to assure proper distribution of the concrete during placement. The underpinning was successful and no settlement occurred thereafter.

After the flood of May 1990, a complete contour of the river bottom was made (Figure No.4) at the bridge site. The deep trench found downstream of Pier 2 was still present but had been partially backfilled with sediments. The survey also showed a depression between the pivot pier and Pier 2. A diver inspection was made of the pivot pier and rest pier which showed both cofferdams in relatively good condition.

On January 1, 1992 at approximately 11:00 PM with the river level again over the top of the piers, a drift removal gang stationed on the bridge reported a loud noise and observed what appeared to be a shift in one of the pivot pier bearings. While additional inspections were being made, both bearings disappeared and the previously noted turbulent water at the pier became calm. Traffic was stopped and detours were immediately arranged since it was obvious that severe damage had occurred. The swing span was now only supported at its ends on rocker bearings. The continuous rails prevented an unstable condition. No excessive deflection was observed in the swing span which now was double its original support length.

To determine the extent of pier damage the profile sonar was used again (Figure No.5). Contours revealed that the cofferdam had sustained extensive damage and had ripped apart on the southeast corner. The opening caused the interior to scour in similar fashion to that experienced at Pier 2. Apparently the pier loading became unbalanced resulting in the masonry break-up. A comparison of river bottom contours of the two floods (Figures No. 4 & 5) showed a significant difference with a very deep trough having developed in the 1992 flood to the downstream side of Pier 2 and between the two piers. That downstream trough was now some 85 - 90 ft deep during the flood.

Prior experience with working on the river was the primary influence on restoration and repair options limited to working with only the existing superstructure and access by rail. As experienced with prior floods, a high river stage was expected for several months and it did occur.

Since the span stayed in position without any damage and there was only about 1.5 inches of deflection, it was felt that it would be worth determining if it was feasible to strengthen the span. A metallurgical test confirmed that the steel was A7 steel, a weldable steel. Calculations were made that confirmed that it was feasible to take the 2-112.5 ft continuous swing truss span and convert it into a single

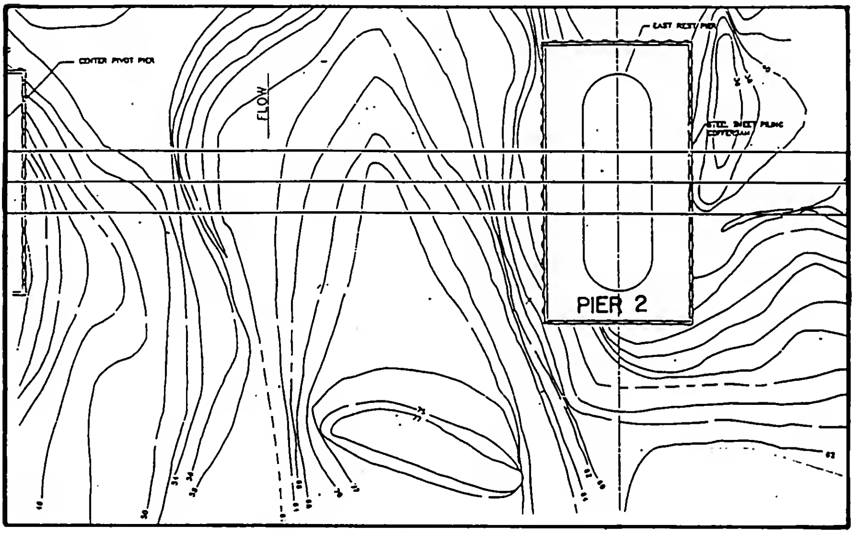

Figure No. 4 – May 1990 River Bottom Contour

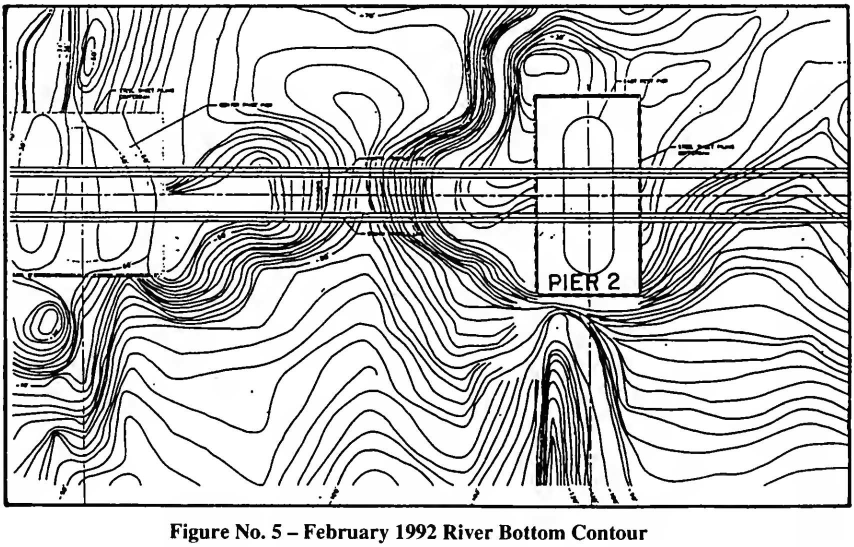

Figure No. 5 – February 1992 River Bottom Contour

225 ft long span (Figure No.6), doubling the span length in-place. The stresses in the span were calculated for the As-Is condition which showed minimal reserve strength in key members. It was also decided that in order to accomplish this conversion to a simple span it would be necessary to deviate from normal design standards. The methodology used rating stress levels. The difference between the member actual stress and allowable stress became the stress level used for sizing the strengthening sections. Added

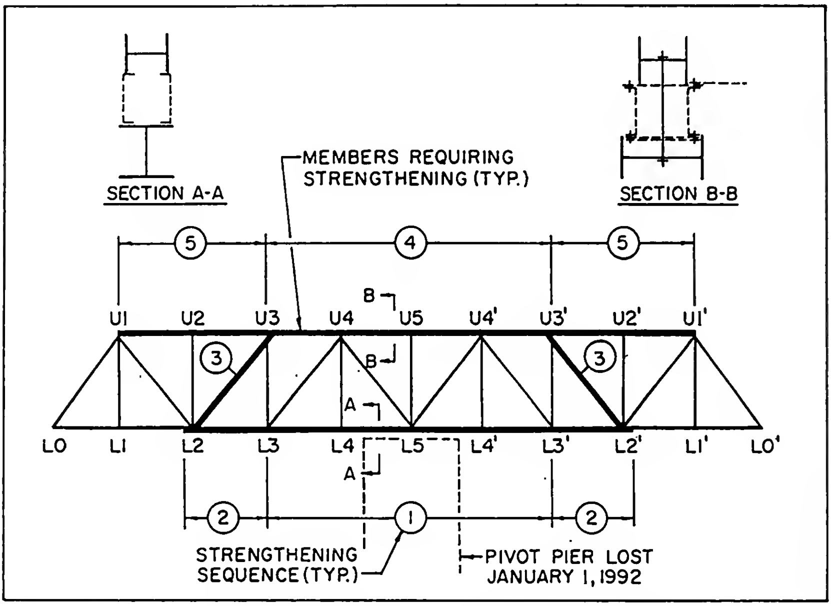

Figure No. 6 – Swing Span Strengthened to 225 Ft. Long Simple Span

members ranged in size from W12X87 to W14X233 sections. The bottom and top chords were "sandiwched" with wide flanged beams welded to the existing built-up riveted shapes. It was necessary to do this to minimize eccentricities in the truss connections. Detailing to minimize overhead welding was done. Lacing bars were removed as needed for placing the added sections in contact with the existing member primary sections. Other strengthening items included several diagonals and several truss connections and converting one set of rockers to fixed bearings.

Although the repairs would be performed under an emergency situation and since the repair design would be pushed to the limit on span capacity, it was felt that a very strict quality control procedure should be followed. Therefore, the contractor was required to submit his bid based on this using the AWS D1.5 Bridge Welding Code, have full-time QC personnel throughout the strengthening process, and have an independent lab examine welds. The Engineers performed QA. The Southern Pacific secured bids and awarded the strengthening contract to H.B. Zachry based not only on their price but primarily on their agreement to complete the strengthening work in 3 weeks. The Contractor was required to work continuously 24 hours per day until completion. The Contractor furnished up to 30 workers per 12 hour shift. For this project the SP established a new base camp which was also open 24 hours a day.

The work sequence was very important and necessary to prevent overloading the truss during the strengthening. The bottom chord underside at the center was strengthened first. Those beams were rolled out on rail carts and cranes located off the span using a flat boom angle assisted in placing them. Those beams were set only inches above the flood waters. Due to the January cold wet weather, tents were built around the welding areas and preheat, interpass heat, rod oven usage, and other welding procedures were strictly observed. Each welder was required to be certified and was required to sign each weld for record. Welding proceeded around the clock until completion in an 18 day period (Figures No. 7 & 8).

A final test was felt necessary to assure that the span was adequately strengthened to its original capacity at double the original span length. It was calculated that the span would deflect 2-1/4 inches under the load of filled hopper cars. A test train (Figure No.9) backed the load onto the span and a deflection of 1-5/8 inches was observed. After multiple runs, the load was left in place for a short period.

Figure No. 7 – Swing Span Prior to 1992 Flood

Figure No. 8 – Swing Span Converted to a 225' Long Simple Span

Figure No. 9 – Loaded Test Train for Checking Deflection of Strengthened Span

Full recovery was observed. The welds were observed after each train for several days and inspection intervals reduced with time. No broken welds were found. A 5 mph slow order was placed on the bridge until a new center pier could be constructed.

After several periods of fluctuating river stages and under a separate contract with Massman Construction Company, a pile bent pier was installed under the span at the former pivot pier location and the east pier was encased in concrete. On October 7, 1992 train speeds were returned to the timetable speed of 30 mph.

In both of the flood damage events, the emergency response was driven by the fact that temporary repairs to allow train movements were required to be done in a high river stage environment. Waiting for a lower stage would have greatly increased the detour cost to an excessive amount and additional damage or structural failure could have occurred during the waiting period. It is estimated that 8 million dollars were saved in detour costs, train delays, and construction cost by taking these bold repair measures. It was the willingness, cooperation and understanding between the parties (Railroad, Contractors and Engineers) that made both emergency responses successful.

NEW 1993 AREA SPECIFICATIONS FOR HIGH-PRESSURE GAS PIPELINES ON RAILROAD RIGHTS OF WAY

By: J. B. Miller*

In 1992 sub-committee 5 of Committee 1- Roadway and Ballast completed assignment D5-3-91 which was to develop specifications for uncased high pressure gas pipelines within the railway right of way. These specifications became the major part of a much needed rewrite in Chapter One of the manual. Also included in the rewrite was a review and update of the existing Part 5 on specifications for pipelines conveying liquid flammable substances, and specifications for pipelines conveying non-flammable substances. These proposed manual changes were approved by the full committee as well as the AREA board and were published in the Bulletin 739 (January 1993) for information and comment. The changes are now incorporated in the latest 1993 -1994 Manual Revisions.

The New Part 5 consists of 3 sections: Specifications for Pipelines conveying Liquid flammable substances,Specifications forUncased Gas Pipelines within theRailway Right of Way, and Specifications for Pipelines Conveying Non-flammableSubstances. These sections have uniform scope articles which have provisions to increase or tighten the specifications when risks increase due to track speed, traffic density, traffic sensitivity, terrain conditions, curvature and grade, bridges and other structures, pipe size, capacity and material carried, and environmental risks. There are cross section drawings for each section with all dimensions and notes referenced to the appropriate article. All three sections have uniform articles covering approval of plans.

The specifications for gas pipelines in Part 5 are based on the design methodology developed by the gas research institute at Cornell University. The design methodology was verified in the field testing of 12" and 36" pipeline installations under the heavy haul loop at FAST. Another section on carrier pipe provides a series of ten minimum nominal wall thickness tables which cover pipe diameters up to 42". Pipe steel specified minimum yield strength (SMYS) from 35000 to 70000psi and maximum allowable operating pressure (MAOP) up to 2000 psi. The table wall thicknesses are based on the following criteria:

- The maximum allowable hoop stress due to internal pressure as specified in regulatory codes
- The maximum combined multiaxial stress due to all external and internal loads
- Fatigue in girth welds due to external live loads
- Fatigue in longitudinal seam welds due to external live loads.

The design parameter assumptions used to calculate the table wall thicknesses are as follows:

- Depth of carrier pipe from base of rail is 10 feet
- Double track condition is assumed
- Modulus of soil reaction E = 500 psi
- Soil resilient modulus Er = 10, 000 psi
- Girth weld is located at centerline of track
- Overbore of 2" over pipe diameter during installation class location design factor F=0.6
- Factor of safety FS = 1.5 used in design criterion

The tables are easy to use and will provide adequate wall thickness for most installations. If actual crossing conditions fall outside these parameters, a detailed analysis must be performed using the design methodology referenced in Part 5 which refers to a commentary on the "Design of uncased pipelines at railroad crossings" and the "Guidelines for Pipelines crossing railroad" which outlines the design

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methodology as developed by Cornell University under the sponsorship of the gas research institute. This commentary was published in the December 1992 AREA Bulletin 738.

The new uncased specifications also provide for special protection when there is a possibility of having foreign materials in the subgrade, unusual potential for third party damage or for other reasons. This special protection may require concrete jackets on the carrier pipe, protection slabs above the pipe or the depth of burial increased. Soil borings may also be required to determine soil characteristics and to identify if foreign material is present in the bore. Depth of burial is also covered in Part 5, and requires that the top of the carrier pipe be at least 10 feet from the base of rail at its closest point. At all other locations on the rights-of-way the minimum ground cover must be 6 feet. Longitudinal gas pipelines must be at least 25 feet from any track and must have a minimum ground cover of 6 feet up to 50 feet from the centerline of track. Longitudinal lines more than 50 feet from the centerline of track must have a minimum ground cover of 5 feet. Longitudinal lines must be marked by an approved sign every 500 feet and at every road crossing, streambed, other utility crossings, and at locations of major change in the direction of the line. The section covering flammable liquid pipelines has these same requirements for longitudinal lines.

While there are locations where the use of casing pipe is the only practical choice, there are also specifications which will provide an uncased option. The design methodology used for the pipe wall thicknesses is very conservative and the 10 foot burial will provide increased protection from third party damage and damage from derailments. The gasline industry seems to be agreeable to the increased burial depth to get rid of the casing pipe and the problems of monitoring cathodic protection within the casing pipe. This specification will also address gas lines installed using directional boring methods as they are usually considerably deeper than 10 feet. Therefore external live loads would not be a factor.

This new specification only covers gas pipelines. The API (American Petroleum Institute) has adopted the GRI design methodology, so it will be only a matter of time before we start getting requests from them and other pipeline operators for uncased crossings. When that time comes we should consider their request and come up with a specification that will be in the best interest of the railroads. If we do not consider their requests, we will run the risk of getting a federal regulation that will not be in our best interest.

TRACK TRANSITION PROBLEMS AND REMEDIES[1]

By: Arnold D. Kerr* and Brian E. Moroney**

SUMMARY

At first, track transitions are defined and then the associated track problems encountered are reviewed. This is followed by a presentation of the mechanics of track transitions, and the postulation of the main principle to be followed when searching for remedies. Next, the measures taken by various railroads to reduce the deterioration near transition points are reviewed. It is shown that these remedies, developed mainly by the trial-and-error approach, fall into three distinct categories. The paper concludes with a discussion of methods for improving the current practices. As examples, analyses for the determination of the necessary stiffness of tie-pads and mats are presented, in order to provide a comfortable ride, prevent the occurrence of large dynamic wheel loads, and reduce track damage at these locations.

INTRODUCTION

Transition regions are locations where a railway track exhibits abrupt changes in vertical stiffness. They usually occur at the abutments of open-deck bridges, where a concrete-tie track changes to a wooden tie track, at the ends of a tunnel, at highway grade crossings, at locations where "rigid" culverts are placed close to the bottom of the ties in a ballasted track, etc.

Track transitions caused by a change of rail cross-section are rare and their effect is generally very small. Therefore, they will not be considered here; although remedies for this case are similar.

Transition regions require frequent maintenance. When neglected, they deteriorate at an accelerated rate (compared to a regular track). This may lead to pumping ballast, swinging or hanging cross-ties, permanent rail deformations, worn track components, and loss of surface and gauge (Track Supervisor, 1983; Buchanan, 1983; Weatherman, 1983; Stanley, 1985; Track Supervisor, 1985). These in turn may lead to delays due to slow orders or create a potential for a derailment. Pictures of damaged transition regions are shown in Fig. 1.

(a) At Bridge

(b) At Road Crossing

Figure. 1 Examples of Track Transitions

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Many railroad engineers are aware intuitively that these problems are caused by a sudden change in the vertical track stiffness, although some others attribute them to poor drainage and weak ballast and subgrade sections in the vicinity of the transitions. Although these latter causes may be contributing to the track transition problems, it is believed that the major cause is an abrupt large change of the vertical track stiffness and the associated dynamic wheel loads.

To clarify these problems, in the following we discuss the mechanics of the track transitions and how they are affected by the moving trains. This should clarify the major reason for the encountered track problems and indicate ways for rational remedies.

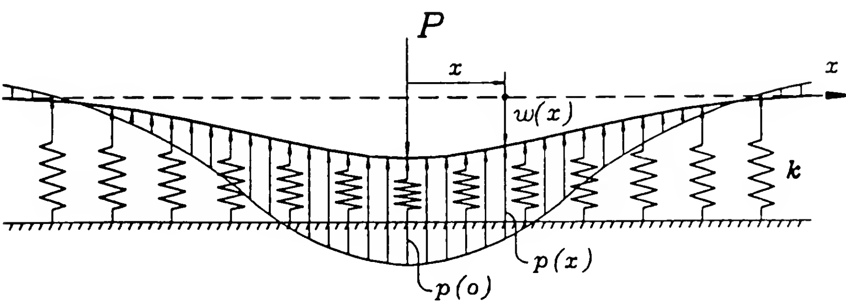


Figure 2 Model for Standard Track Analysis

THE MECHANICS OF TRACK TRANSITIONS

The standard analysis for ballasted cross-tie tracks utilizes a model consisting of two rails which are continuously supported by closely spaced springs (Winkler foundation), as shown in Fig. 2. In this model the springs represent the response of the cross-ties, fasteners, ballast and subgrade. According to this model, the "continuous" contact pressure between a rail and the cross-ties is assumed to be

$$p(x) = k\, w(x) \ , \qquad (1)$$

where w(x) is the vertical deflection of the rail axis at point x, and k is the proportionality factor, referred to in the railroad literature as the "rail support modulus" or the "track modulus". Note, that according to (1), the stiffer the track the larger is k.

Next, consider the track transitions shown in Fig. 3. Also shown is the schematic k-distribution. For well maintained main-line tracks on wooden cross-ties the track modulus is $k = 20$ N/mm^2 ($\cong$3,000 lb/in^2). For tracks on concrete cross-ties it is in the range of 40 to 100 N/mm^2 (6,000 to 15,000 lb/in^2); depending on the stiffness of the ties and the condition of the base. For a track in a tunnel or over a bridge abutment the corresponding k-value may be much larger.

At transition points, because of the abrupt change in the vertical track stiffness, a wheel experiences a rapid change in elevation (either up or down). This in turn causes vertical accelerations of the moving car. According to Newton's law this leads to vertical wheel force changes, due to the resulting dynamics of the moving car. The magnitude of these force changes depends on the elevation difference (rail deflection) between each side of the transition, how k varies in the transition zone, on the velocity of the moving train, and on the suspension characteristics and the masses of the car. The distribution of the vertical force, that a wheel exerts on the rail, depends also on the direction of the moving train. This is shown, schematically, in Fig. 4.

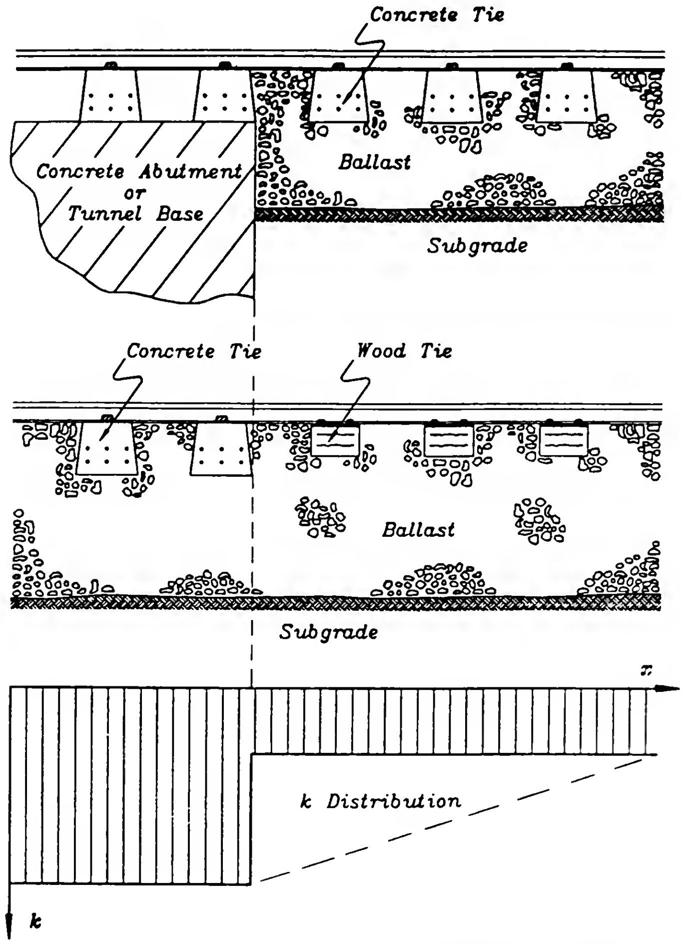


Figure 3. Track Transitions and the Corresponding k-Distribution

In case A, the train is moving toward the bridge. The increased dynamic wheel force, that the rail is subject to, is caused by the lifting of the wheel (and car) up onto the rigid abutment in a very short period of time (a second or less). The region of increased dynamic wheel loads is located at the abutment, as shown. The resulting damage may be battered rails and plate-cut wood ties on top of the abutment near the transition. Also, these impact forces may cause damage to the abutment.

In case B, the wheel is moving off the bridge and onto the softer ballasted track. Because the track-in-ballast deflects much more than the track on the abutment, the wheel "drops" off the abutment imparting increased dynamic forces to the rails, as shown in Fig. 4, case B. The location of the largest force will depend on the train speed; the larger the speed the farther away from the abutment is the maximum vertical wheel force. Since it takes less force to disturb the ballast and the subgrade than it takes to cause rail and tie batter, these wheel loads cause fouled ballast, hanging cross-ties, permanent rail deformations, and the well known "dip" in the track at the end of the bridge. (Fig. 1a)

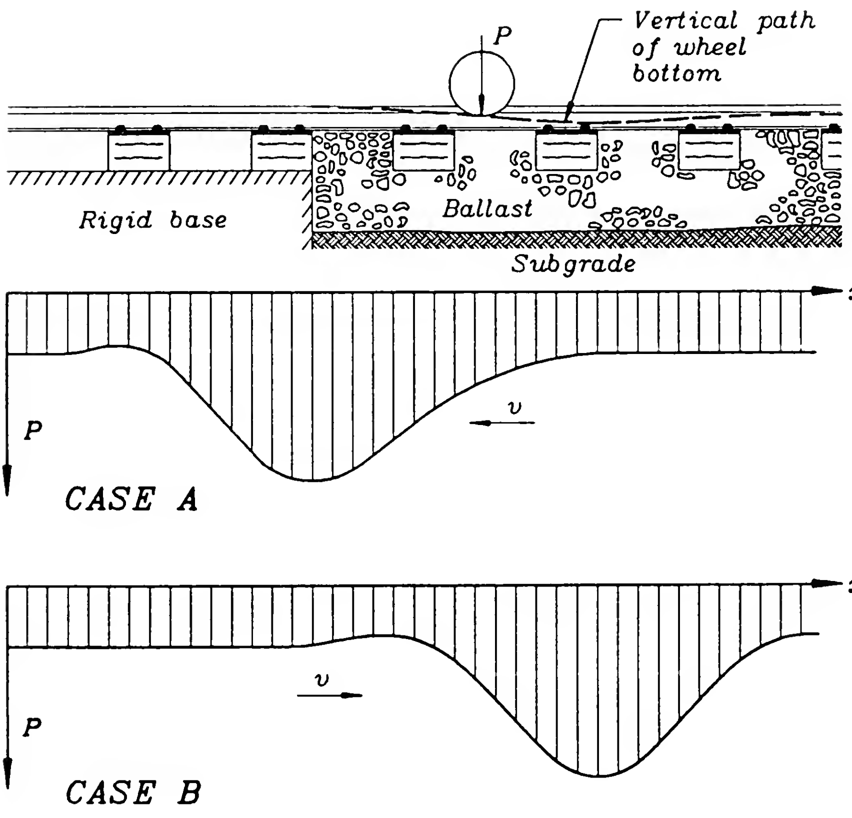


Figure 4. Schematic Dynamic Wheel Force Distribution in Transition Zone

Similar situations may occur at the ends of a tunnel, especially where the ties in the tunnel rest on a hard base, or at both ends of a road-crossing (Fig. 1b) at which the track is stiffened by the road-crossing structure and the stiff ballast-subgrade base, that has been well compacted by the moving auto traffic. Problems of this type may also occur at the transition points between wood and concrete tie tracks, when the track moduli, k, differ substantially, as shown in Fig. 3. In this case the generated dynamic wheel forces of the cars that move toward the concrete tie section may crack the first few concrete ties which adjoin the transition point (and soften the "stiff" side). The result is a "zipper" effect which creates its own transition to the stiffer concrete-tie section.

The above discussion suggests that the main principle to be followed when searching for transition point remedies is: *(a) to assure that the tracks be of such design that the wheels of the moving train cause the same vertical rail deflections, or, (b) if this is not possible, at least the vertical rail deflections should not undergo rapid changes, in order to avoid large vertical accelerations of the wheels (and hence of the cars), which cause large dynamic wheel forces.*

Point (a) requires that the vertical stiffnesses of the track at each of the two sides of a transition point be the same and suggests, where necessary, the use of tie-pads placed above the ties (for wood-tie tracks under the tie plates) or mats placed under the ties or the ballast in the stiffer track section. Point (b) requires the "smoothing" of the transition between the relatively stiff track and the regular soft track. This may be achieved by smoothing the transition of the track modulus k (shown as a dashed line in Fig. 3), or by stiffening of the rail-tie structure on the soft side of the transition point. It should be noted that the needed length of the smoothing region depends on the dominant train speed; the higher the speed the longer the smoothing region.

In the next section the measures taken by various railroads to reduce the deterioration near the transition points, and to improve the ride quality, are presented. Since they were developed mainly heuristically, and often independently, using the trial-and-error approach, they will be correlated with the principle presented above, in order to gain a better understanding of the problems under consideration.

MEASURES USED BY VARIOUS RAILROADS

Different measures have been developed and used by a number of railroads in order to reduce the damage that occurs at transition points. They all have the effect of reducing the vertical accelerations of the wheels and cars as the train moves over the transition zone. These remedies may be grouped in three categories:

(I) The smoothing of the k-distribution on the "soft" side of the transition, as indicated by the dashed line in Fig. 3,

(II) The smoothing of the transition by increasing the bending stiffness of the rail-tie structure on the "soft" side in the close vicinity of the transition point, and

(III) The reduction of the vertical stiffness on the "hard" side of the transition.

When considering category (I), it appears that the most well known remedy adopted by North American railroads is the one which uses oversized ties at the transition point, as presented in the AREA Portfolio of Trackwork Plans (1952) and shown in Fig. 5.

A variation of this approach is being extensively utilized by the Canadian National for open deck bridges. It was later adopted for the transition between wood-tie and concrete-tie tracks, as shown in Fig. 6.

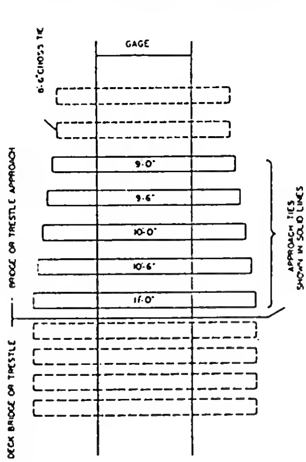


Figure 5 Bridge Approach Transition (According to AREA Portfolio 1952).

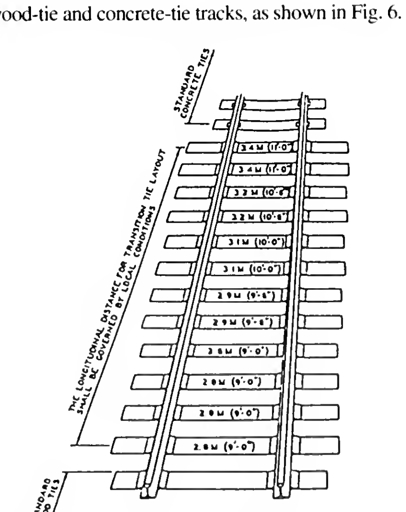
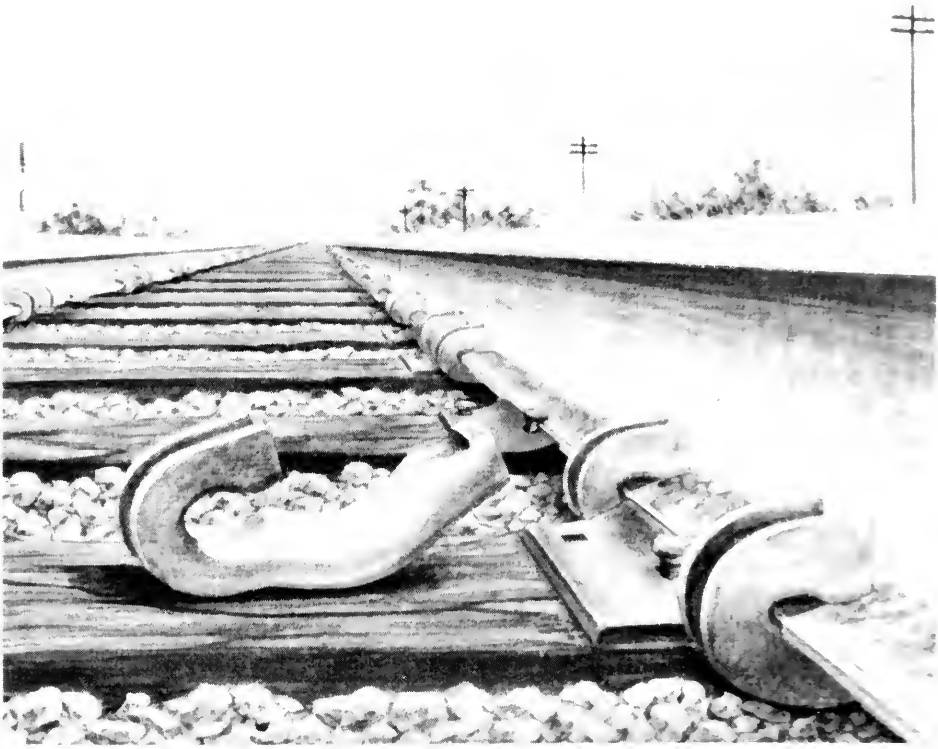


Figure 6 Transition Section Between Wooden-Tie and Concrete-Tie Tracks. (CN MOW Manual SPC 3303, 1981).



Note that this solution utilizes a series of progressively longer wood ties (usually switch ties) to create a more gradual increase in vertical track stiffness on the "soft" side of the transition. The purpose of using longer ties is that by engaging a larger section of ballast they produce a stiffer track. It is therefore essential that these approach ties be kept fully tamped between the end of the tie and a point about 45 cm (1.5 feet) inside the running rail, in order to develop the full available bearing area of these ties (Worth, 1992), otherwise their effectiveness is reduced. Regarding the length limitation of the transition ties, for the same tie cross-section beyond a certain tie length track stiffness does not increase because the tie ends bend upwards.

Also, because embankments of bridge approaches are often very narrow, a tie longer than eleven feet generally may not be provided with a sufficient ballast section to ensure support at the end of these ties (Worth, 1978).

Similar transition sections were adopted by a number of other North American railroads, like the Burlington Northern, CSX Transportation, and Union Pacific. They are presented in the thesis by Moroney (1991).

Another group of remedies which fall into category (1), are those which use geotextiles. Early designs were described by Selby (1981) and by Leubke (1982), as shown in Fig. 7.

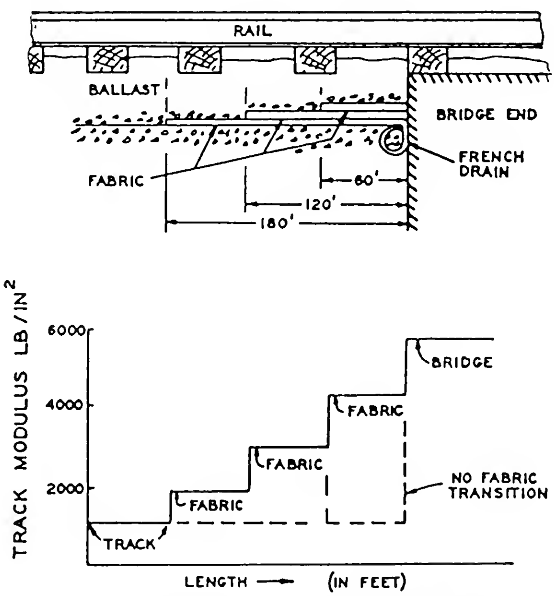


Figure 7. Vertical Stiffness Transition Using Geofabrics (Leubke 1982)

In this method the transition is formed by using three layers of fabric of decreasing length: 150, 100, and 50 ft. by Selby (1981) and 180, 120, and 60 ft. by Leubke (1982).

Note that a horizontally placed fabric has to stretch substantially before it contributes to reinforcing the subgrade in the vertical plane. Therefore, there is a possibility that the observed increase in the vertical track stiffness near the abutment is caused mainly by the fabric's ability to facilitate good lateral drainage resulting in an increase of the track modulus; thus, a stiffer transition track near the abutment

(Raymond, 1986). Because of the varying degree of success achieved to date using geotextiles at transitions (and the question posed above regarding the reason for the generated vertical stiffness increase by horizontally placed geotextiles), more careful testing is needed to establish the suitability of using geotextiles for problems of this type and, if yes, the most beneficial arrangement of the geotextiles.

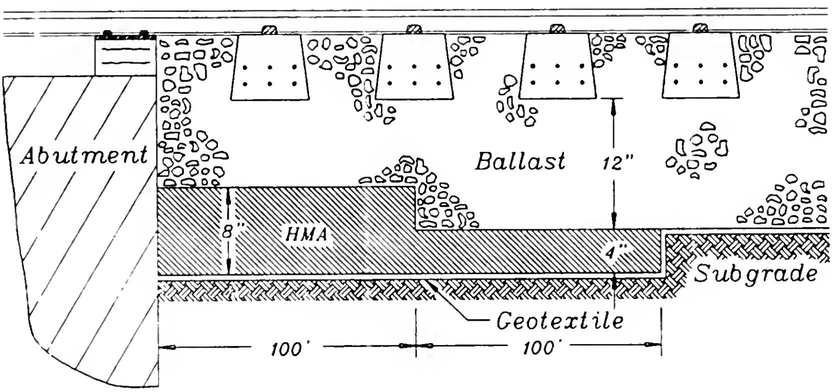
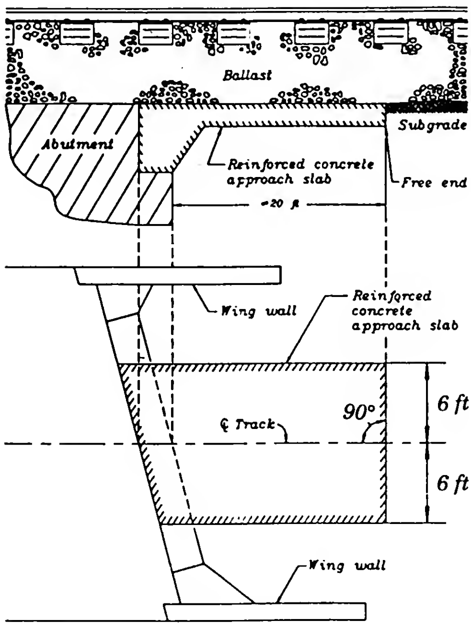


Figure 8. HMA Bridge Approach Transition.

Another method that falls into category (I) uses Hot-Mix-Asphalt (HMA). According to Rose and Hensley (1991) and Rose (1992) a number of tests were installed on the CSX railroad during this past decade. An example is shown in Fig. 8. According to Rose (1992) the primary use of HMA underlayments in transition sections is for rehabilitation of those locations that have exhibited poor performance using conventional procedures, and that their performance monitored to date is very encouraging.

Still another method that falls into category (I), used mainly at bridge abutments or at tunnel ends where slab-track meets cross-tie track in ballast, utilizes a cantilevered slab, one end of which is supported by the "rigid" structure and the other is free, as shown in Fig. 9(a). In addition to providing a stiffness transition, the approach slabs may be used to "square-off" bridge abutments which are located on a severe skew as shown in Fig. 9(b), in order to reduce the possibility of derailment from rocking of rail cars (Chapin, 1990). This method, is similar to the one often used at highway bridge approaches.

A method belonging to category II, which is based on increasing the bending stiffness of the rail-tie structure on the "soft" side near the transition point, is shown in Fig. 10. It was developed by the German Federal Railways (DB) for the ICE high speed lines. The shown installation is at the portal of the Mühlberg tunnel (Schrewe, 1987), where the slab section inside the tunnel changes to a concrete cross-tie track in ballast. The transition section is formed by four extra rails attached to the ties; two between and two outside the running rails. The length of this transition zone is about 30 meters (~100 ft.) long.



(a)

(b)

Figure 9. Transition at Bridge Abutment Utilizing a Reinforced Concrete Approach Slab.

The use of tie-pads falls into category III. The aim is to reduce the stiffness of the track on the "hard" side of the transition point. Ideally, it should match the stiffness of the track on the "soft" side of the transition point.

According to von Schrenk (1928) during the 1870's the French Eastern Railway started to use pads between the rails and ties, to protect the ties from wear. At first they used felt pads but then switched to poplar wood pads (shims) because of better performance and lower cost. Since about 1914, the pre-compressed poplar pads were made a standard over the entire system of the French Eastern Railway (p. 74). A cross-section of the used rail support is shown in Fig. 11(a).

In 1906 some German railroads started to experiment with tracks that used French-type fasteners with poplar wood pads, as shown in Fig. 11(a). Note that the fastener did not use a metal tie-plate and that the size of the poplar pad was the same as the contact area between the rail base and tie. As pointed out by Schwemann (1920) tracks with fasteners of this type exhibited almost no rail creep.

The notion of utilizing elastic pads under rails which are placed directly on a solid structure (like a bridge abutment), for reducing the impact forces of the moving trains and thus diminish the damage to the "rigid" base, is described by Müller (1925, 1928). A fastener of this type with a poplar wood pad is shown in Fig. 11(b). Later, poplar pads were also utilized on steel bridges and under turnouts for force reduction and sound attenuation purposes .

Figure 10. Transition Section Which Utilizes Additional Rails, DB, (Schrewe, 1987).

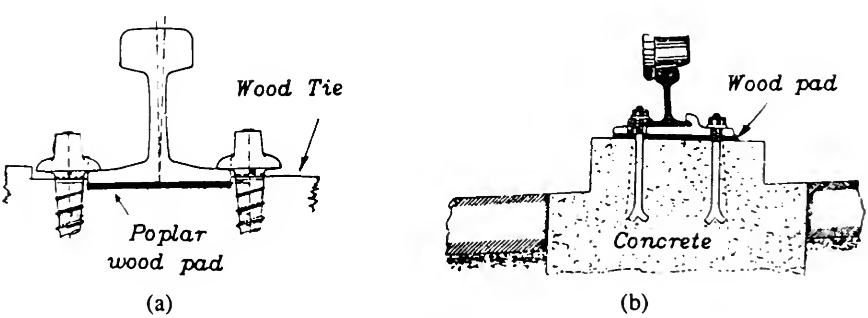


Figure 11. Early Utilization of Pads.
(a) French Eastern Railway, von Schrenk (1928); (b) German Railway, Müller (1925).

The K-fastener introduced by the German Railways in 1928 incorporated the poplar wood pad, that was placed between the rail base and the tie-plate, as a standard feature. It's main purpose has been to increase the resistance against axial rail push-through, thus to strongly reduce rail creep. This eliminated the need for anchors. According to Müller (1930) this pad also facilitates the rotation of the rail over the tie-plate, thus diminishing the high edge contact pressure between the tie-plate and tie and reducing the rotation of the tie about its axis; this in turn reduces tie bottom abrasion. In the recent past, the wooden pads in the K-fastener have been replaced by elastomeric pads (Weiss, 1981).

The effect of tie-pads on track were studied for many years in North America by the AAR under the sponsorship of AREA Committee 5. The aim of these studies was to reduce wood tie abrasion. The results were published in a series of progress reports in the AREA Proceedings, Volumes 48-64, from 1947 to 1963. Lateral motions of the tie-plates on the ties and the effects of water underneath the tie-plates were considered to be the important factors that cause wood-tie abrasion and plate cutting. Therefore, these tests focused mainly on utilizing tie-pads to cushion against the lateral motion of the tie-plates and to prevent water from reaching the tie fibers directly under the tie-plates.*) This damage mechanism – the lateral movement of the tie plates on the ties – was discussed in Germany before and after WWI and led to the introduction of the K-fastener in 1928, which provides for a strong attachment between tie-plate and wood tie, using four screw spikes. For related discussions in the USA refer to von Schrenk (1928).

Magee (1953, p. 1408) recommended that tie-pads be used to protect bridge ties and joint ties. It appears that while those areas do tend to experience excessive plate cutting and tie wear, it is the result of increased dynamic loads on the ties, in addition to the motion of the tie-plates over the ties and the effect of water between the tie-plates and the ties.

The tie-pad test areas of the AREA-AAR study were located on the Louisville & Nashville Railroad. Inspections of the test sections were made on an annual basis and the observations reported in the AREA Proceedings. In its final report the AREA (1967, p. 364) concluded that: (1) Tie-pads protect the ties from plate cutting, but only in the pine ties did they show any result to indicate that an increase in tie life may be expected from their use, (2) Tie-pads appeared to have little effect on gauge in tangent track, but had a tendency to contribute to gauge widening in curves, and (3) Tie-pads should be no less than 3/16-in. (4.8 mm) thick and of such material that they withstand a static tensile test of 1250 lb (5,560 N) longitudinally and 500 lb (2,224 N) transversely.

It is of interest to note that although the AREA study recommended a tie-pad thickness and its tensile strength, it did not discuss the necessary vertical stiffness, nor how this may affect the dynamic vertical forces at the rail seat.

Currently pads are being utilized in connection with concrete ties and spring-clip fasteners throughout the world. In addition to increasing the rail push-through resistance, and hence the elimination of the need for anchors, the purpose of the pads is to attenuate the impact forces of the wheels, in order to prevent damaging the concrete ties. The necessary stiffness of these pads depends on the condition of the track and the rolling stock (Dean et al., 1983; ORE D161.1, Report 3, 1986; Reinschmidt, 1991). This problem is as yet not completely solved. For studies on pads in the Russian language, refer to Shakhunyants and Demidov (1971), Shul'ga and Bolotin (1975, 1977), and Shakhunyants, Demidov, and Gasanov (1978).

According to Kaess and Schultheiss (1986), the German Railways (DB) have been experimenting on the ICE-high speed lines with softer pads in tunnels and on bridges. The aim of these tests was to reduce the vertical track stiffness at these locations and to match it with the tracks outside. Since on some sections of the German high-speed lines there are many tunnels and bridges, it was hoped to create a uniform track stiffness throughout, thus provide a comfortable ride and at the same time prevent the

*) Note the analogy with the recent discussions and a devised remedy related to rail-seat abrasion of concrete ties. (Reinschmidt, 1991)

occurrence of large dynamic wheel forces. According to Kaess (1992) these tests revealed that the ICE tracks are rather stiff and that pads will have to be used throughout. For a related discussion refer to Leykauf and Mattner (1990). The use of pads on bridges and in tunnels, for the purpose of reducing large variations of the vertical track stiffness, were discussed by Bussert and Bothe (1978) for regular rail traffic on the railways of East Germany.

In the recent past, pads have been also utilized at transition points by some North American railroads. For example, according to Worth (1992), "tie pads are standard on wood ties on open deck bridges on CN (since the early 1980's) wherever annual tonnage exceeds about two MGT annually. Their primary purpose is to seal water out of the spike holes and reduce iron sickness under the plates, but they also serve to put some resilience into the system." In the United States, one railroad that uses pads is the CSX (Maintenance Rules and Practices, 1988, page 57). According to Hardy (1989), tie-pads are being utilized by CSX on open deck bridges to reduce mechanical wear of the bridge ties. At CSX mostly pine or softwood are used for bridge ties "...and the pad greatly reduces plate cutting. The pad material is solid neoprene, 1/4 inch thick and about the size of the tie plate. It is held in place by the spikes that hold the plate and rail." CSX does not prescribe the stiffness of the pad.

For the past decade, tie pads have been utilized also in grade crossings. An example is shown in Fig. 12. According to the Hi-Rail company crossing brochure, "...It is essential to use a tie pad under all tie plates in crossing area and extended to at least 3 ties beyond each end." No specific pad stiffness is prescribed.

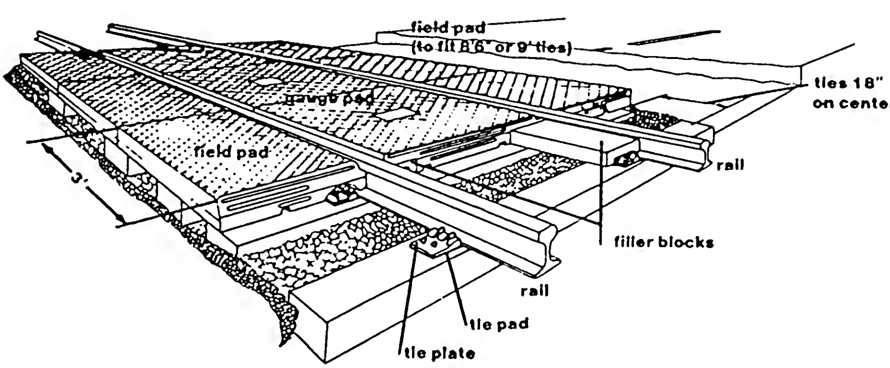


Figure 12. Hi-Rail Grade Crossing with Tie-Pads (from company brochure).

The above review on the use of tie-pads, and the previously established principle for track transitions, point to a need for prescribing a proper vertical spring coefficient for the pads to be used on the "stiff" side of the rail-tie structure. This is necessary in order to secure a uniform vertical stiffness throughout the track section under consideration or, at least, to minimize the stiffness differences along the transition. A simple method to achieve this aim will be presented in the next section.

A remedy which falls into category III is based on the utilization of mats. According to Sato, Usami and Satoh (1974), in order to increase the train speeds to 250 km/h (155 mph) on the Shinkansen network, the notion was conceived to insert rubber mats between the ballast and the solid base of the elevated structures and in tunnels. The main purpose of this ballast-mat was to reduce the dynamic wheel loads and thus to prevent the "pulverization" of the ballast, which was occurring in the tracks of the Takaido Shinkansen.

The ballast-mat was produced of used automobile tires that were cut into fine grains and heat molded into a mat. This procedure was found suitable for producing the desired spring constant by adjusting the voids in the mat. Numerous field and laboratory tests, initiated in 1971, were conducted on the produced mats, regarding the necessary mat stiffness, as well as mat strength, durability, etc. A major finding of this extensive testing program was that ballast-mats were highly effective for preventing ballast pulverization. They also reduced noise and vibrations. For details of this study refer to Tajima and Kiura (1974), Sato and Usami (1977), and Sato, Kobayashi, Nakamura, and Kobayashi (1983). According to Sato and Usami, by 1977 ballast-mats have been widely used on the Japanese National Railways.

In Germany the possible use of elastic mats was mentioned by Eisenmann and Leykauf (1977). Keim, Kohler and Schober (1978) presented test results for Sylomer™-mats. Another ballast mat, Vibrex™, which consists mainly of natural rubber with fabric reinforcement, is described by Clouth (no date). Kaess and Stretz (1979) described an extensive study on mats of various types conducted by the German Railways (DB), regarding their necessary properties. In this connection note the recent Deutsche Bundesbahn (1988) specifications for ballast-mats.

In concluding this review it should be noted that the rubber boot and microcellular pad, surrounding each of the two blocks of a two-block tie that is embedded in a very stiff base, shown in Fig. 13, may be considered as a mat or a pad, since the two blocks of the tie are relatively rigid.

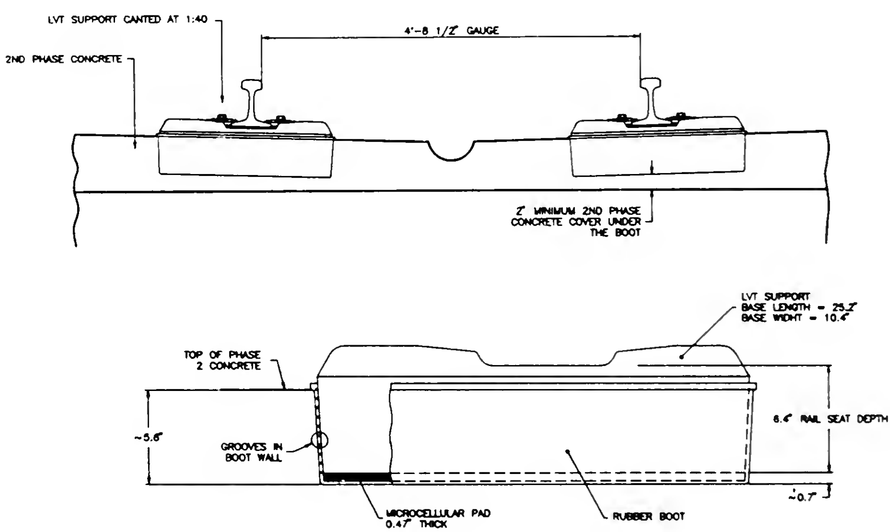


Figure 13. Two-block Tie with Rubber Boot and Microcellular Pad. (Sonneville International Corp. 1992)

THE DETERMINATION OF THE PROPER STIFFNESS OF PADS AND MATS

Determination of the Needed Pad Stiffness

The test for determining if a chosen pad stiffness is correct consists of three steps: (1) the tie-pads are inserted into the transition area, (2) the trains are allowed to move over it at regular speed, and (3) the vertical accelerations of the passing cars and the dynamic forces that the wheels exert on the rails are recorded. If the results are not satisfactory, the pads are to be changed and the test run again. This was apparently done by the DB on the ICE-high speed lines (Kaess and Schultheiss, 1986, p. 37).

To choose the initial value of the pad stiffness one may use a pad that is available, or guess the needed stiffness, or determine the stiffness analytically. In the following it will be shown, how the needed pad spring coefficient may be calculated in an attempt to produce a track of uniform vertical stiffness. The first example is a concrete-tie track which enters a tunnel where it rests on a "rigid" base, similar to the case shown in the upper part of Fig. 3. The corresponding analytical models for concrete as well as wood ties are shown in Fig. 14.

In the tunnel the rail is supported discretely with pad spring coefficient κ, as shown in Fig. 15(A), whereas outside the support is represented continuously with the rail support modulus k, as shown in Fig. 15(B).

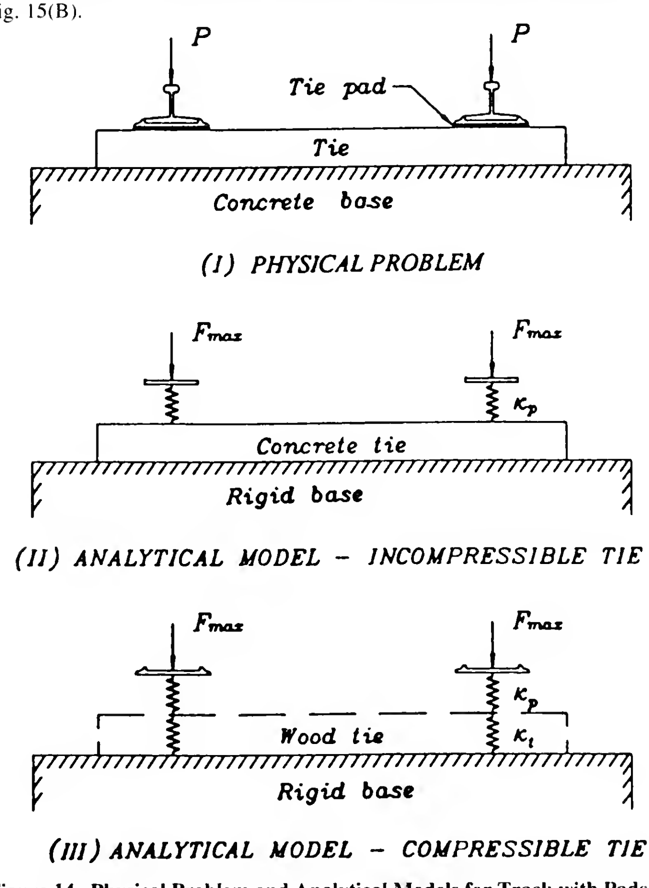


Figure 14. Physical Problem and Analytical Models for Track with Pads.

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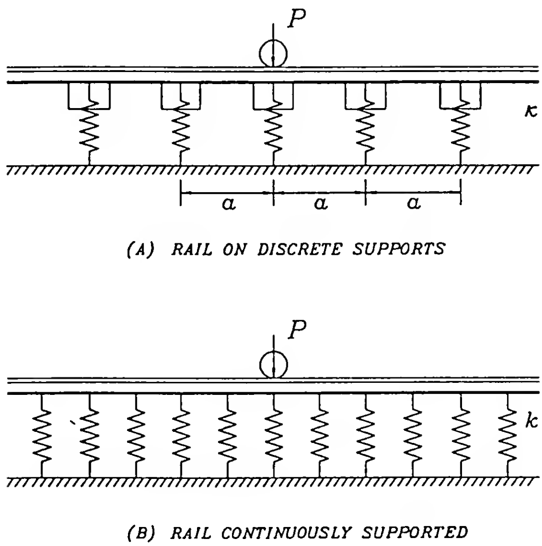

Figure 15. Discretely and Continuously Supported Track Models.

In order to match these two tracks, the relation between κ and k has to be established. This is done next, following the derivations by Kerr (1976).

Vertical equilibrium of a rail on discrete and on continuous supports, as shown in Fig. 15, requires that

$$P = \sum_{n=-\infty}^{\infty} \kappa w_n = \kappa \sum_{n=-\infty}^{\infty} w_n \tag{2}$$

and

$$P = \int_{-\infty}^{\infty} p(x)\,dx = k \int_{-\infty}^{\infty} w(x)\,dx \tag{2'}$$

noting that according to eq. (1), p = kw, and κ and k are the constant track parameters for one rail.

Since in the above equilibrium equations the wheel load P is the same, the right-hand-sides may be equated. Thus,

$$\kappa \sum_{n=-\infty}^{\infty} w_n = k \int_{-\infty}^{\infty} w(x)\,dx \tag{3}$$

The integral in the above equation is the area formed by the deflection curve. Multiplying eq. (3) by the center-to-center tie spacing, *a*, and noting that for small *a*

$$\sum_{n=-\infty}^{\infty} w_n a \cong \int_{-\infty}^{\infty} w(x)\,dx \tag{4}$$

eq. (3) reduced to

$$\kappa = a \bullet k\,. \tag{5}$$

This is the relationship between the spring constant κ of the discrete support and the rail support modulus k.

For the case when the "incompressible" concrete ties rest on a rigid base, as shown in Fig. 14 (II), κ is the spring constant of the pad only. However, for a wooden tie track, as shown in Fig. 14 (III), the κ in eq. (5) represents the equivalent spring constant for the tie-pad and the compressible wooden tie. When the response of these two track components is *linear*, with the parameters κ_p and κ_t, the equivalent spring constant is

$$\kappa = \frac{\kappa_p \bullet \kappa_t}{\kappa_p + \kappa_t} \tag{6}$$

To select the appropriate tie-pad for a tunnel or abutment, at first the actual k-value of the adjoining track is to be determined. This may be done using the simple method suggested by Kerr (1983, 1987). Then the necessary pad spring constant, κ, is calculated using eq. (5).

As *first example*, consider the case of a concrete tie track that enters a tunnel with the rails continuing over a relatively rigid concrete slab. There, it is planned to place the rails on tie-plates in order to increase the contact area. To compensate for the missing ballast and subgrade, softer pads are to be inserted between the tie-plates and the concrete base. The task is to determine the necessary pad stiffness in order to have a nearly uniform k-value throughout.

For the adjoining track it was determined that k = 40 N/mm^2 (6,000 lb/in^2). The rail support spacing is a = 61 cm (24 inches). Therefore, the required "spring constant" for each pad in the tunnel is, according to eq. (5),

$$\kappa = a \bullet k = 610 \times 40 = 24{,}400 \text{ N/mm} \quad (140{,}000 \text{ lb/in}) \tag{7}$$

Thus, the thickness of the pad should contract by 1 mm when subjected to a force of 24.4 kN.

If the κ-values for the available tie-pads are not known, they may be determined using the test shown in Fig. 16. The size of the tie-pad should be the same as the size of the rail-tie contact area; for the present example, it is the size of the tie-plate, 14"x8". The testing machine is exerting a force F, through the rail and tie-plate, on the pad. The force F (representing the rail seat force) is increased from zero to about 1/3 of the anticipated largest wheel load [the largest rail seat force, Kerr (1981)]. While the load is increasing, the corresponding vertical displacement of the metal tie-plate is recorded for each load. The κ-value of the pad is determined from these results.

Tests were conducted using a 1/4" thick Alert™ tie-pad.* Since a wheel load of a 100 ton car is about 30,000 lb., the loading range was chosen from 0 to 12,000 lb. The recorded results are shown in Fig. 17, as curve (1). Because this curve is non-linear, it was approximated by a line, as shown. Noting that for the linearized case $F_o = \kappa w_o$, the corresponding κ value is

$$\kappa_p = \frac{F_o}{w_o} = \frac{10{,}000}{0.05} = 200{,}000 \text{ lb/in} = 35{,}025 \text{ N/mm} \tag{8}$$

* An Alert™ tie-pad is made of milled rubber and fibers, cast and vulcanized in slabs of prescribed thickness and then cut to the desired shape.

Comparing this pad stiffness with the required $\kappa = 24{,}400$ N/mm determined in eq. (7), it follows that the tested single pad, although it substantially reduces the stiffness over the rigid support, is too stiff and a softer pad should be used. When the rails are placed directly on the concrete base, without tie-plates, the pads will be smaller and a new test curve (I) has to be produced.

As a *second example*, the track on oak wood-ties (shown in Fig. 4) is considered with k = 20 N/mm^2 (3,000 lb/in^2). The corresponding κ is, according to eq. (5),

$$\kappa = a \cdot k = 610 \cdot 20 = 12{,}200 \text{ N/mm } (70{,}000 \text{ lb/in}) \tag{7'}$$

To soften the track over the "rigid" abutment, or over the base of the tunnel liner, pads are to be inserted over the wood ties. Next, the necessary stiffness of these pads is to be established. At first, the spring constant of the oak tie was determined experimentally using the test set-up shown in Fig. 16 by replacing the pad with an oak tie. The recorded load vs. vertical displacement values are shown in Fig. 17, as curve (II). It is of interest to note that the recorded stiffness of the tested tie is smaller than that of the tie-pad given by curve (I). It's linear spring constant is

$$\kappa_t = \frac{F_o}{w_o} = \frac{10,000}{0.07} = 142,857 \text{lb/in} = 25,018 \text{N/mm} \tag{9}$$

Next, the pad and the tie were tested, by inserting a wood-tie under the pad in the set-up shown in Fig. 16. The recorded results are presented in Fig. 17 as curve (III). The corresponding spring constant is

$$\kappa_{pt} = \frac{F_o}{w_o} = \frac{10,000}{0.12} = 83,333 \text{lb/in} = 14,594 \text{N/mm} \tag{10}$$

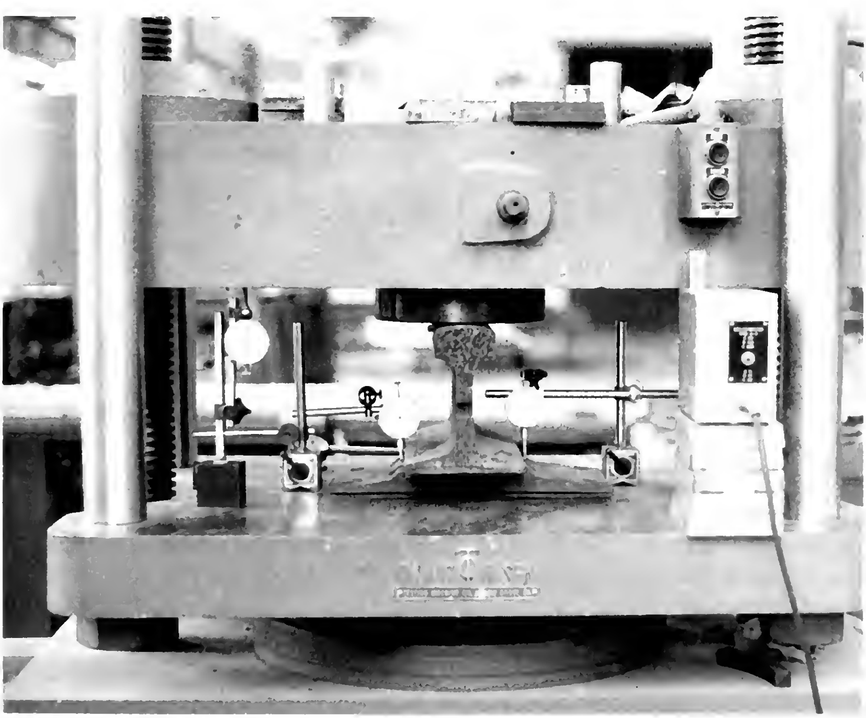

Figure 16. Test Set-Up for Determination of κ_p.

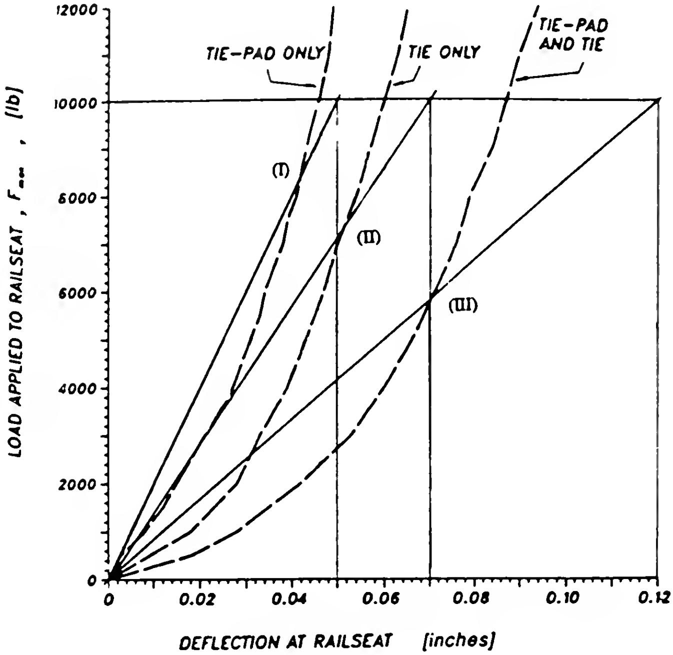

Figure17. Recorded Load-Deflection Test Curves.

-------------------- Recorded, —————— Linearized

This value is larger than the required calculated value $\kappa = 12{,}200$ N/mm, presented in (7'). Thus the chosen pad should be slightly softer.

The κ_{pt}-value in eq. (10) was determined from test curve (III). As a check, it is calculated from eq. (6). The result is

$$\kappa_{pt} = \frac{35,025 \bullet 25,018}{35,025 + 25,018} = 14,594 \text{ N/mm} \tag{11}$$

It is same value as the one determined from the test curve (III) and given in (10). However, this exact agreement is purely coincidental, since it depends on the way the curves in Fig. 17 were linearized. Generally κ_{pt} in eq. (11) should be larger than the one given in eq. (10), because a solid pad resting on a "porous" wooden surface appears softer than when it is confined between two metal plates (curve I). Therefore, in general, curve (III) is better suited for the determination of the linear spring constant κ_{pt} for pad and tie. In this connection it should be noted that some pad materials are nearly incompressible and therefore these pads should be of such form as to allow it to compress vertically (shape factor).

As a *third example*, consider a concrete tie track, with wood-ties over the "rigid" abutment, as shown in Fig. 8 (without the Hot-Mix-Asphalt layer). For the adjoining concrete-tie track the spring constant κ is given in eq. (7), as $\kappa = 24{,}400$ N/mm (140,000 lb/in). For the wood-ties over the abutment the corresponding spring constant is given in eq. (9), as $\kappa_t = 25{,}018$ N/mm (142,851 lb/in). Thus, the stiffness of the wood-tie tested is nearly sufficient for matching the stiffness of the adjoining concrete-tie track.

The main purpose of the above numerical examples is to demonstrate the method for estimating the necessary pad stiffness. The presented analysis determines only a *preliminary* value, since the used method is valid for very slow speeds; namely when the dynamics of the moving train and track are negligible. Also, for high speed trains the loading and unloading of the pads occur very rapidly. For this reason, when establishing the pad stiffness, the pad tests have to be conducted for loading rates that will be encountered in the field.

The next step is to insert the chosen pads in the "stiff" track section and then record the dynamic wheel loads which are caused by the moving trains, in the transition region. If the recorded wheel force variations are too large, the pads in track should be replaced with pads of a different stiffness and the test run again. The performance in revenue service yields the final proof as to whether the proper pad stiffness was chosen.

Determination of the Needed Mat Stiffness

Next consider the case when a concrete tie track is placed directly on a "rigid" bridge butment, or on the concrete invert of a tunnel (similar to the situation shown in the upper part of Fig. 3), and a mat is placed between the ties and the relatively rigid base, in order to create a uniform stiffness through the transition zone. The resulting physical problem and the corresponding analytical models are shown in Fig. 18 (I) and (II).

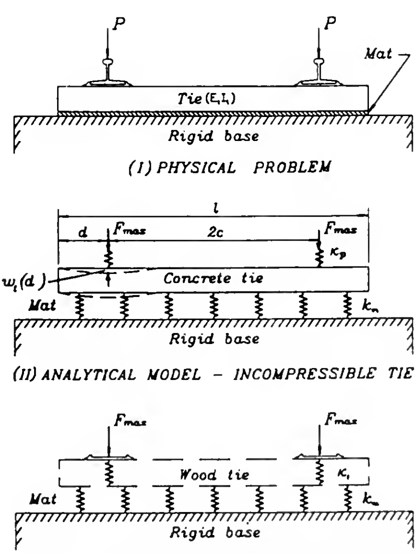

Figure 18. Physical Problem and Analytical Models for Ties on Mat.

Assuming that the mat response may be represented by a linear elastic Winkler foundation with parameter k_m, it follows (Hetényi 1946, p. 56) that the largest rail seat force F_{max}, will cause the tie deflection at the rail seat

$$w_t(d) = \frac{F_{max}\beta_o}{2k_m} \cdot \frac{B}{\sinh(\beta_0 l) + \sin(\beta_o l)} \tag{12}$$

where

$$\beta_o = \sqrt[4]{k_m/(4E_tI_t)} \tag{13}$$

$$\begin{aligned} B = {} & 2\cosh^2(\beta_o d)[\cos(2\beta_o c) + \cosh(\beta_o l)] \\ + {} & 2\cos^2(\beta_o d)[\cosh(2\beta_o c) + \cos(\beta_o l)] \\ + {} & \sinh(2\beta_o d)[\sin(2\beta_o c) - \sinh(\beta_o l)] \\ - {} & \sin(2\beta_o d)[\sinh(2\beta_o c) - \sin(\beta_o l)], \end{aligned} \tag{14}$$

k_m is the vertical stiffness of the mat per unit length of the tie axis, E_tI_t is the bending stiffness of a cross-tie in the vertical plane, and c, d, l are shown in Fig. 18.

Next, noting Fig. 18, it follows that

$$F_{max} = \kappa_m \, w_t(d) \tag{15}$$

where κ_m is the equivalent discrete spring constant of the mat and tie at the rail seat. From above relation it follows that $\kappa_m = F_{max}/w_t(d)$, and noting (12),

$$\kappa_m = \frac{2k_m}{\beta_o} \cdot \frac{\sinh(\beta_o l) + \sin(\beta_o l)}{B} \tag{16}$$

Next, noting that the "springs" of the tie-pad and mat and tie are coupled in series, it follows that the resulting equivalent spring constant is

$$\kappa_{total} = \frac{\kappa_p \bullet \kappa_m}{\kappa_p + \kappa_m} \tag{17}$$

In order to match the rail on discrete supports with parameter κ_{total} and the rail on "continuous" support with parameter k, we use eq. (5). Namely

$$\kappa_{total} = a \bullet k \tag{18}$$

Equating the right hand sides of eq. (17) and (18), it follows that the expression for the determination of k_m is

$$k = \frac{\kappa_p \kappa_m}{a(\kappa_p + \kappa_m)} \tag{19}$$

where κ_p is the spring constant of the rail-tie pad, κ_m is the equivalent discrete spring constant of the mat and tie at the rail seat given in (16), and $\beta_o = \sqrt[4]{k_m/(4E_tI_t)}$

To simplify the use of eq. (19) for the determination of k_m, it was evaluated numerically for the center-to-center tie spacing a = 61 cm (24 in), $E_tI_t = 10^8$ kN•cm^2 ($3.48 \cdot 10^9$ lb•in^2) for a concrete tie, and tie dimensions l = 275 cm (108 in), c = 76 cm (30 in), d = 75 cm (29.5 in). The evaluation was performed by substituting chosen values of k_m and κ_p = 200, 400, or 1,000 kN/mm into eq. (19) and calculating the corresponding k-values. The results are presented graphically in Fig. 19.

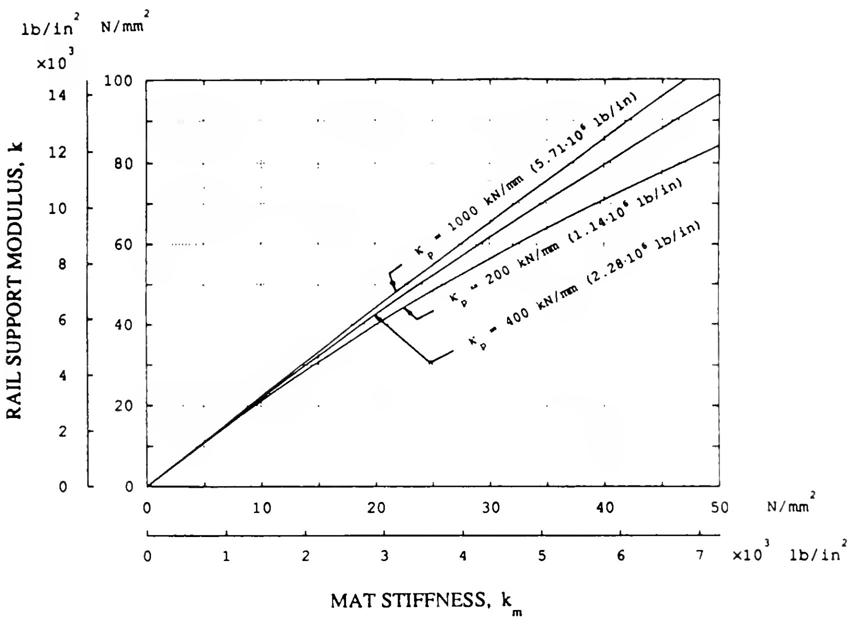

Figure 19. Graphs for Evaluation of k_m-values Based on Eq. (19)

As an example, consider a concrete-tie track in ballast with the rail support modulus k = 40 N/mm2 ($\cong$ 6,000 lb/in^2), and a given pad of stiffness κ_p = 200 kN/mm ($1.14 \cdot 10^6$ lb/in). According to Fig. 19, the necessary mat stiffness over the "rigid" structure is k_m = 20.0 N/mm^2 (3,000 lb/in^2). Next, a mat has to be chosen with this stiffness. To establish the k_m-value of a given mat, a test similar to the one described for a pad is to be conducted.

It is of interest to note that when the concrete ties are considered to be "rigid", thus when $E_tI_t \to \infty$, then $\beta o \to 0$, $\beta o / [\sinh(\beta_o l) + \sin(\beta_o l)] \to 1/(2l)$, $B \to 8$, the expression in (12) reduces to

$$w_t = \frac{2F_{max}}{k_m l} \qquad (20)$$

or rewritten

$$F_{max} = \left(\frac{k_m l}{2}\right) w_t \qquad (21)$$

Therefore, the corresponding mat spring constant is

$$\kappa_m = \frac{k_m l}{2} \tag{22}$$

as expected. The number 2 in the denominator appears because eq. (12) refers to one rail only, and so does the mat spring constant in (22).

The expression that corresponds to eq. (19) is, noting (17), (18) and (22),

$$k = \frac{1}{a} \cdot \frac{\kappa_p k_m l/2}{\kappa_p + k_m l/2} \tag{23}$$

To show the effect of tie bending stiffness, $E_t I_t$, on the necessary mat stiffness, k_m, the expressions in (19) and (23) were evaluated for various tie and pad parameters. The results are presented graphically in Fig. 20.

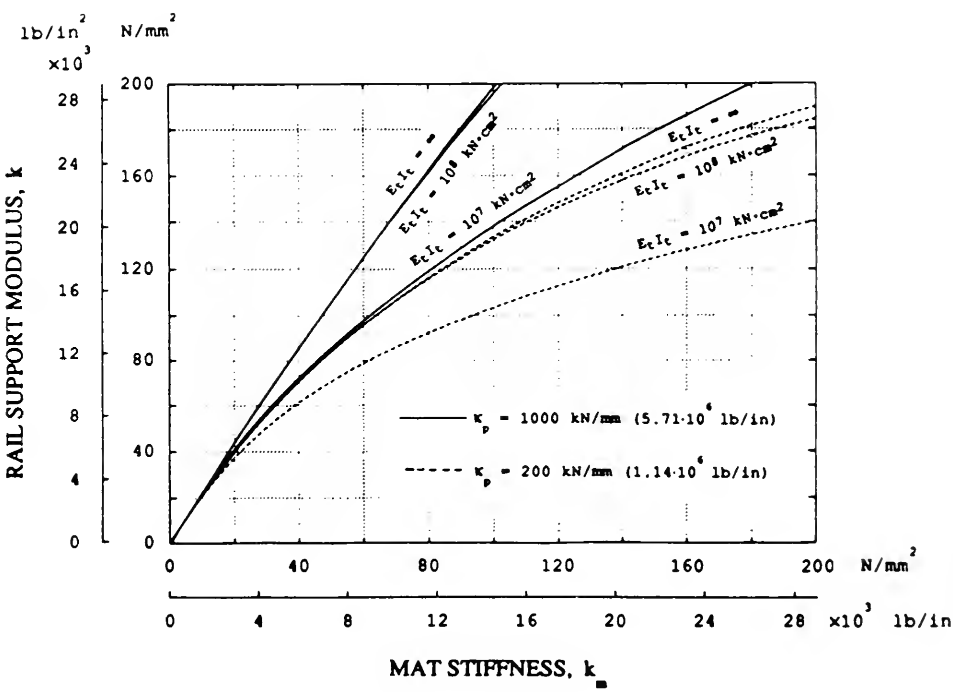

Figure 20. Effect of Tie Bending Stiffness $E_t I_t$ on the determination of k_m.

Note that the curves for concrete ties with $E_tI_t = 10^8$ kN•cm^2 (3.48 • 10^9 lb•in^2) and κ_p of 200 and 1,000 kN/mm agree very closely with the curves for $E_tI_t = \infty$ ("rigid" ties) for the shown range of the rail support modulus k, which covers concrete tie tracks currently in use on main lines. This justifies the use of the simpler expression in (23). Solving it for k_m, results in the simple formula

$$k_m = \frac{2\kappa_p a k}{l(\kappa_p - a k)} \tag{24}$$

For comparison purposes also the graphs for a lighter tie, with $E_tI_t = 10^7$ kN•cm^2, (3.48 • 10^8 lb•in^2) are shown.

In North America for a wood-tie track, pads are generally not used. The corresponding analytical model is shown in Fig. 18 (III). The determination of the needed mat stiffness is essentially the same, by noting that for this case the κ_p in the above analysis is replaced by the equivalent κ_t of tie compressibility, and E_tI_t is the bending stiffness of the wood ties in the vertical plane. When pads are used, like in the K-fastener, then all the three "springs", with their parameters κ_p, κ_t, and κ_m, are acting in series. The resulting equivalent spring parameter is then

$$\kappa = \frac{\kappa_p \kappa_t \kappa_m}{\kappa_t \kappa_p + \kappa_m \kappa_p + \kappa_t \kappa_m} \tag{25}$$

Equating the right side of (25) with the right side of (18) yields the equation for the determination of k_m

$$k = \frac{\kappa_p \kappa_t \kappa_m}{a(\kappa_t \kappa_p + \kappa_m \kappa_p + \kappa_t \kappa_m)} \tag{26}$$

Eq. (26) is analogous to (19). The procedure for the determination of k_m is the same.

Each of the presented analyses determines only a preliminary value for the needed mat stiffness, since the used analysis is valid for ties of uniform cross-section and very slow speeds (when the dynamics of the moving train and the track are negligible and the tie is in contact with the mat over the entire length of the tie). Therefore, the next step is to insert the chosen mats under the ties in the "stiff" track section and then record the dynamic wheel loads that are caused in the transition region by the moving trains. If the variations of the wheel load are too large, the mats under the ties should be replaced with ones of different compressibility and the resulting track structure tested again. This procedure is continued until a suitable mat stiffness is established.

CONCLUSIONS

At first, track transition problems were defined. Then, the mechanics of transition points was presented and the main principles, to be followed when searching for remedies, were postulated. It was shown that the reviewed remedies, devised by various railroads in Europe, Japan and North America, mainly by a trial-and-error approach, fall into three distinct categories: (I) the smoothing of the k-distribution on the "soft" side of the transition, (II) the smoothing of the transition on the "soft" side" by increasing the vertical bending stiffness of the rail-tie structure, and (III) the reduction of the vertical stiffness on the "hard" side of the transition by using pads and mats. Analytical methods for estimating the needed stiffness of a pad and/or mat were presented and their use demonstrated on numerical examples.

As wheel loads and train speeds continue to increase, it is expected that problems in track transition zones will become more frequent and more severe. It is therefore recommended that railroads initiate systematic testing programs to establish which of the remedies are the most suitable for their use (mechanically and economically), and the optimal design parameters for a chosen remedy.

ACKNOWLEDGEMENTS

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CONTROLLING RAIL AND WHEEL WEAR ON COMMUTER OPERATIONS

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Introduction

The Regional Railroad Division of the Southeastern Pennsylvania Transportation Authority (SEPTA) operates approximately 450 route miles of commuter operation in the Philadelphia metropolitan area. In 1991, the railroad division began experiencing an unusually high rate of wheel and rail wear, particularly on sharp curves on the commuter rail lines. This wear was predominately wheel flange/rail gage face wear with an apparent correlation between the increase in wear on both the wheels and the rails.

The cause of the increase in wear was not clear or well defined, and as a result the most appropriate form(s) of corrective action was difficult to determine. In order to effectively address this issue, a comprehensive examination of the wear mechanisms, wheel and rail, was undertaken, together with a detailed study of the dynamic interaction between the wheel and the rail. As part of this detailed study, a series of specific activity areas were defined, and each of these areas were addressed both individually and as part of the overall integrated study.

These specific study areas included the following:

1. Field investigation of rail and wheel wear.
2. Computer simulation of wheel/rail interaction and wear.
3. Analysis of wheel and rail profiles.
4. Analysis of rail grinding effects and requirements.
5. Evaluation of rail lubrication effectiveness.
6. Evaluation of track geometry (super-elevation and unbalance) effects.
7. Assessment of critical measurements and tolerances.
8. Economic benefit analysis.

This paper presents a summary of each of these activities together with some of the overall finding and recommendations of this study. It should be noted here that this study was specific to the conditions and operations of the Railroad Division of SEPTA, and any extrapolation of these results to other properties should be done with extreme caution.

Field Investigation of Rail and Wheel Wear

Severe rail wear was observed and measured by SEPTA's Regional Railroad Division particularly on track with moderate to severe curvature. This observed rail wear was primarily gage face wear, with the specific mode of rail wear being abrasive wear as manifested by the rough surface of the rail gage face and the presence of visible metallic particles on the base of the rail, tie plates, etc.

In an effort to define and evaluate the rail wear, a series of wear measurements were taken on a broad range of curves within the Railroad Division. Rail wear measurements were made using a hand gauge at multiple locations on each measured curve. Figure 1 presents some representative measurement data, converted to units of rail wear (in inches/MGT). Specifically, Figure 1 presents the gage face wear rate as a function of curvature for the measured rails. Note: while there appears to be a trend towards the

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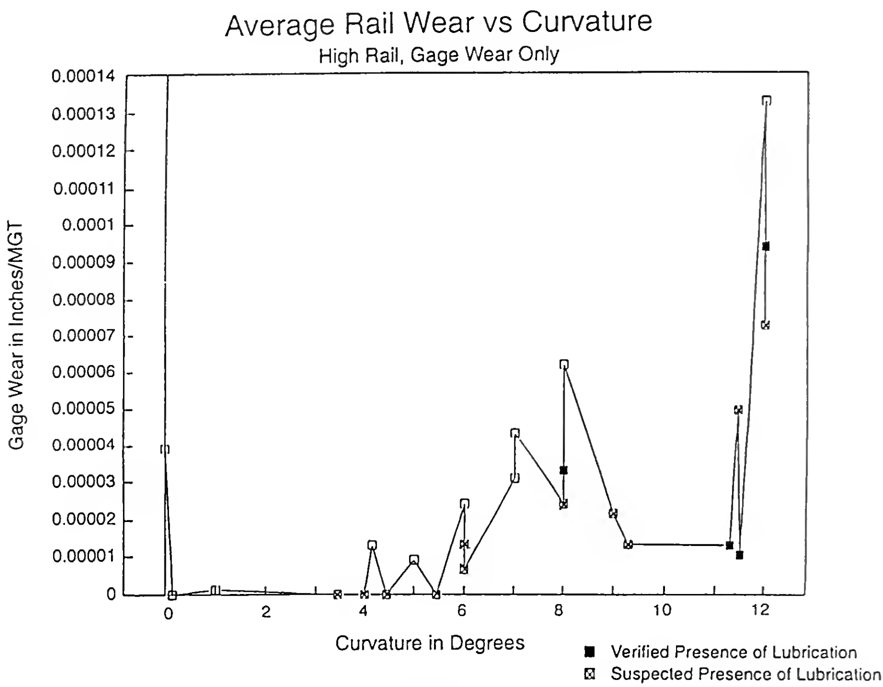

Figure 1. Measured Rail Wear

expected rate of wear increase with curvature, there are several well defined points which show either more or less wear than expected. Several of these points were determined to be locations where rail lubrication was present. Analysis of the data, combined with inspection of the track and rail conditions, determined that non-uniform lubrication practice was a major cause of the variation in the rail wear rate.

In a manner analogous to that of the rail, severe wheel rail wear was also experienced by SEPTA's Regional Railroad Division. This had manifested itself in extremely short periods between wheel truing and wheel replacement.

The primary mode of the wheel wear was flange wear, and appeared to correspond to the high rate of gage face wear observed on the rail.

In order to analyze the wheel wear, the set of cars that operated in a dedicated service. i.e. the SEPTA airport line, were examined using wheel measurement data maintained by SEPTA during the car's regularly scheduled inspections.

Of the nine cars that normally operated on the airport lines, six of the cars had sufficient data as to allow for a detailed wheel wear analysis. Figure 2 presents the wear data from one of the cars, number 234, during the period between December 1988 and August 1991. It can be clearly seen in this Figure that the wear rate during the last 18 months (March 1990 to August 1991) was significantly greater than that during the first 15 months (December 1988 through April 1990). The other five cars similarly exhibited high wear rates during 1990-1991, corresponding to Figure 2.

Detailed Vehicle-Track Curving Model

After the initial determination of the high incidence of wheel and rail wear, an analysis of the wheel/rail curving interaction was performed using ZETA-TECH Associates, Inc.'s VEHMOD vehicle-track curving model. This theoretical model has the capability of analyzing the curving behavior of a complete transit vehicle around a range of curvatures, and was used to examine the wheel/rail interaction as a function of the key track and vehicle parameters, to include both wheel and rail profiles.

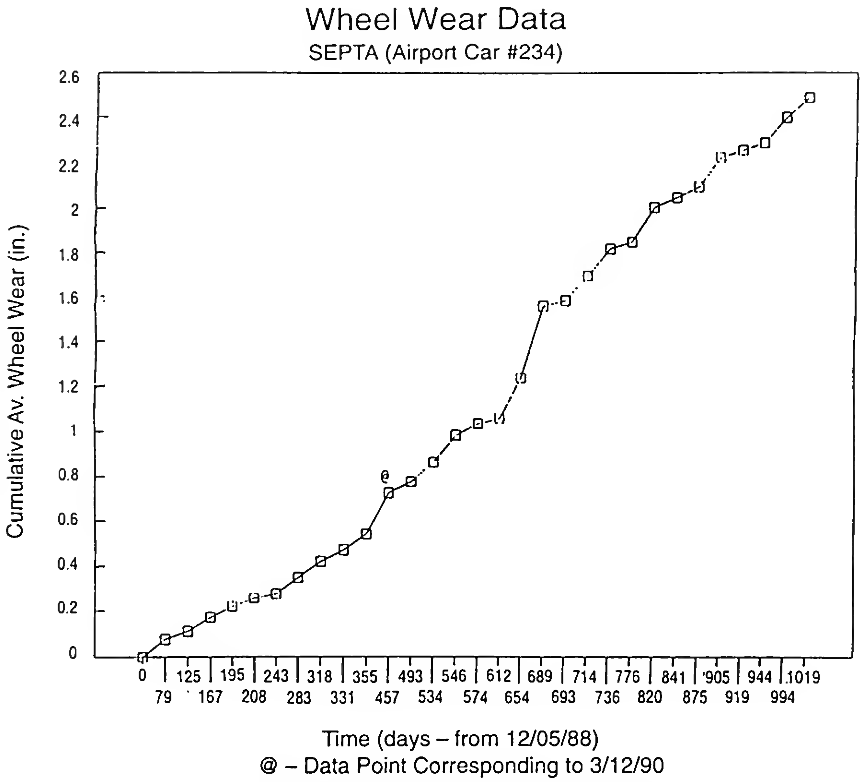

Figure 2. Wheel Wear Data: (Car No. 234, 12/88-8/91

The model calculates the following interim and final output parameters:

- –Wheel/rail forces
- –Energy dissipated at flange (α flange wear)
- –Energy dissipated at tread (α tread wear)
- –Forces between key car/truck components

The model is sensitive to all key design and operating parameters including:

- –Car design
- –Component stiffness
- –Component geometry/tolerances
- –Track geometry (curvature, elevation, ...)
- –Speed
- –Spiral geometry
- –Wheel and rail profiles

The model was used to simulate operation of SEPTA car equipment across a full range of curvatures, wheel and rail profiles, and other key variable parameters. The specific results of each set of analyses will be discussed in the appropriate section of this paper.

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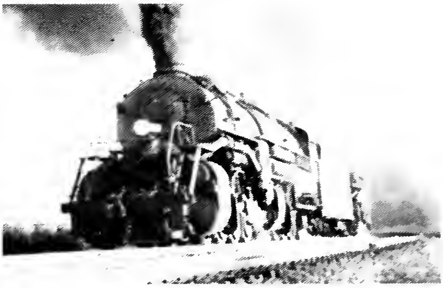

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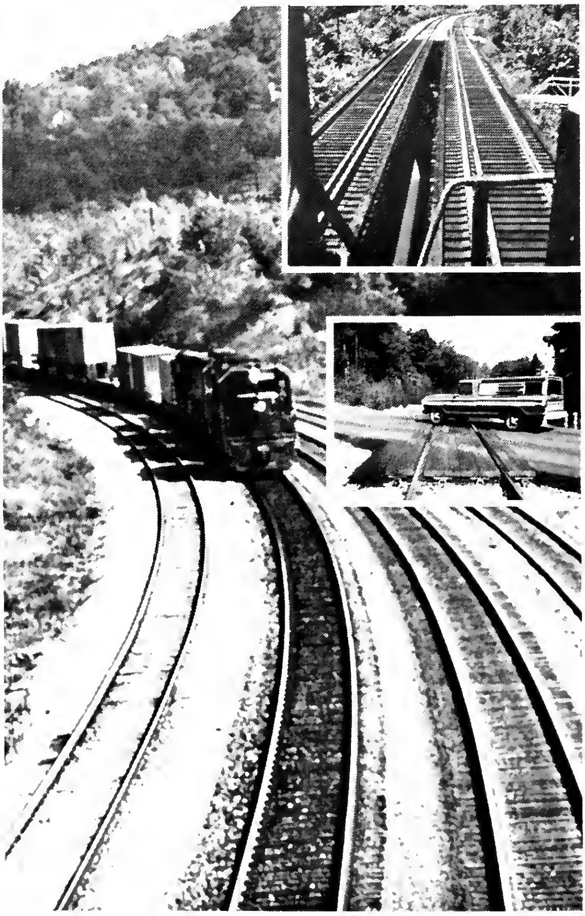

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Analysis of Wheel/Rail Profiles

One key area of evaluation of wheel/rail dynamic interaction, wheel/rail forces, and resulting wear, was the effect of the profiles, both wheel and rail.

In the case of the rail, the concept of rail profile control (by rail grinding) was examined from the point of view of controlling rail wear, and in particular rail gage face wear. The resulting analysis indicated that this approach of rail profile control by grinding was in fact appropriate for SEPTA, particularly on shallow to moderate curves. For more severe curves, where flanging will occur, it was determined that this same approach was appropriate, but should be augmented with a one point contact between the flange of the wheel and the gage corner of the rail.

The specific profile recommendations for SEPTA included separate profiles for tangent track, high and low rails on shallow curves (3 degrees or less), and high and low rails on sharp curves.

In all cases, a rail head radius of between 8 and 10 inches was defined. In the case of tangent track, a centered contact zone, approximately 1 1/4" to 1 1/2" in width, was defined on both rails. For the shallow curves (3 degrees or less), where flanging could be controlled, separate profiles for high and low rails were defined. For the high rail, the contact zone was set to be 0.4" to 0.6" in width located to the gage side of the rail center. For the low rail, the contact zone was set to be 0.50" to 0.75" in width located to the field side of the rail centerline.

For sharp curves, greater than 3 degrees, again separate profiles were defined. On the high rail, where flanging will occur, one point contact was determined to be the most effective. The contact band on the high rail was correspondingly set to be an approximately 1.5" zone on the gage corner of the rail. (Note that the width changes slightly for new or worn rail.) For the low rail, the same contact zone as presented for shallow curves was to be maintained.

In a manner analogous to that of the rail, the SEPTA wheel profiles were examined from the point of view of minimizing wheel/rail wear. This analysis was performed using the wheel/rail curving model described previously.

SEPTA's current wheel profiles are standard AAR 1:20 profiles. This profile was compared to the newly developed AAR 1B profiles, from the point of view of minimization of wheel/rail contact energy at the interface between the wheel flange and the gage face of the rail.

Figure 3 presents a comparison of the energy dissipated at the wheel flange for these two profiles (AAR 1:20 vs AAR 1B). As can be seen from this figure, the AAR 1B profile shows a reduced level of energy dissipation for a wide range of curvatures. This translates into a lower wheel/rail wear rate.

Based on these model results and additional model analysis, it was determined that The AAR-1B profile improved the effective conicity of the wheels, thus improving steering of the wheelsets. This then reduces flanging on moderate curves. Based on this, the AAR-1B profile was recommended for control of wheel-rail wear on SEPTA. However, it was noted that the AAR-1B is more susceptible to hunting. While hunting is not currently a concern for SEPTA's present equipment, it was noted that hunting or potential hunting situations should be monitored during the initial introduction of the 1B profile to insure that no new problems develop.

Rail Grinding

Rail grinding was carefully examined from the point of view of controlling and maintaining the rail head profile (see previous discussion) as well as for control of surface defects on the rail itself. Grinding has come into extensive use on railway systems to control rail surface defects such as corrugations, battered welds, engine burns, etc, as well as being part of an overall rail maintenance approach, based on the control of the wheel/rail contact zones through the grinding of special "profiles" onto the rail head. This latter use has a direct application in the control of both rail and wheel wear, as already noted.

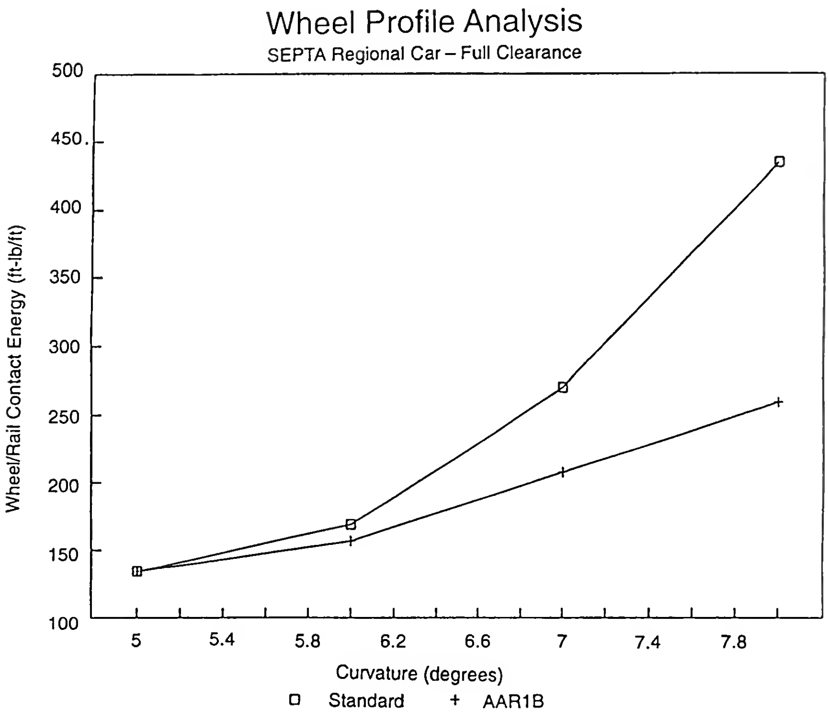

Figure 3. Comparison of Wheel/Rail Contact Energy for Different Wheel Profiles

Rail grinding was therefore used for both profile control and control of surface defects. In the former case, rail profile grinding was defined as the approach required to establish the recommended rail head profiles. Noting that any rail profile will deteriorate under traffic, it is necessary to periodically monitor the profile and to regrind the rail head (maintain the profile) when the profile deteriorates to the point where it is no longer functioning properly. If this profile maintenance is not carried out (i.e., if the profile is allowed to deteriorate and is not restored), then the benefits of the ground profiles will no longer continue. Consequently, profile grinding must be considered an ongoing maintenance activity with periodic "maintenance" grinding required to retain the optimum rail head profile.

Based on SEPTA's traffic levels and measured rail condition, it was determined that profile grinding to restore and maintain the optimum rail profile is required one to two times a year.

In addition to profile control, grinding is also used to control surface defects such as corrugations, battered welds, engine burns, and other rail surface defects. Based on the analysis of SEPTA's rail conditions, a series of recommendations for controlling surface defects by grinding was developed, see Table 1.

In addition, the required annual grinding program for SEPTA's Railroad Division was estimated to be of the order of 360 pass miles per year, based on conventional grinding (defects) only, and 540 pass miles per year, based on profile grinding (which was the recommended approach). These estimates were based on the use of a small 20 motor grinding machine that has full motor adjustability. As part of this program, a formal rail grinding planning and evaluation procedure was developed consisting of the following steps:

a. Pre-grind evaluation

b. Procedures for grinding equipment

c. Intermediate evaluation

d. Evaluation of grinding effectiveness

e. Use of measurement tools

TABLE 1

Rail Grinding Standards for Surface Defects

- Corrugations

  –Grind when 50% of corrugations .012 -.015"
  –Depth after grinding < 0.005"

- Weld Conditions

  –High welds grind when 25% > .015"
  –Height after grinding < 0.005"
  –Low welds ground when 25% > .025"
  –Depth after grinding < 0.010"
  –Low welds > .040" should be welded

- Other Surface Defects

  –Grind engine burns when 50% > .020-.025"
  –Depth after grinding < .010"
  –Engine burns > .040" should be welded
  –Grind new rail within 20 MGT of installation

Rail Lubrication

Lubrication plays an important role in the reduction of rail and wheel wear, and the extension of rail life in high wear locations on track. Experience on other properties, as well as controlled test data, has shown that the wear life of the rail can be increased from two to ten times with effective lubrication.

SEPTA has been using a mix of wayside lubricators and hand lubrication for its rail lubrication needs. The wayside lubricators, which are distributed throughout the system, were frequently turned off because of concern for the lubricant (rail grease) getting to the top of the rail head and causing difficulties in train braking. The use of hand lubrication was intermittent and manpower intensive and did not appear (based on observations and measurement) to be applied in sufficient quantity and frequency to properly lubricate the system, given the level of traffic and degree of rail and wheel wear.

Analysis of SEPTA's lubrication practices indicated that it could be improved significantly to extend the wear life of rails in curves. It was estimated that more "effective" rail lubrication practices could extend rail wear life to two or more times that of reported curve rail life, in many cases. However, in order to achieve this level of improvement, a proper combination of lubrication techniques was required. This included replacement of the hand lubrication with more effective lubrication techniques, such as a hi-rail mounted system for lubricating curves or a more comprehensive wayside lubricator system. Such a hi-rail vehicle could apply a layer of lubricant, accurately to the gage face of the rail, continuously along the curves. Alternately, a more extensive wayside lubrication system was required, operating at a high level of effectiveness. In order to accomplish this, a strong emphasis has to be placed on lubricator maintenance with the use of dedicated lubricator maintainers.

In all cases, the level of lubrication was to be carefully monitored to insure that an adequate level of lubricant was obtained, while minimizing the movement of lubricant to the top of the rail head.

Superelevation and Unbalance

Wheel/rail forces, and particularly wheel/rail lateral forces, will increase as a function of increasing overbalance or increasing superelevation. This is clearly illustrated in Table 2, which shows the relative effect of unbalance on wheel/rail forces for a broad range of curvatures. Since wheel flange wear and rail gage face wear are directly related to the wheel/rail lateral forces, decreasing the amount of allowable unbalance will reduce the level of wear, all other factors constant.

TABLE 2
Effect of Unbalance

	Unbalance (Inches)					
Curve	0	1	2	3	4	5
1	1.00	1.08	1.17	1.25	1.34	1.42
2	1.00	1.06	1.12	1.18	1.24	1.30
3	1.00	1.05	1.10	1.15	1.19	1.24
4	1.00	1.04	1.08	1.13	1.17	1.21
5	1.00	1.04	1.08	1.11	1.15	1.19
6	1.00	1.03	1.07	1.10	1.14	1.17
7	1.00	1.03	1.06	1.09	1.12	1.15
8	1.00	1.03	1.06	1.09	1.12	1.15
9	1.00	1.03	1.06	1.08	1.11	1.14
10	1.00	1.03	1.05	1.08	1.11	1.13

However, unbalance permits the operation of higher speeds on track within a defined safety envelope. Thus, reducing allowable unbalance will directly reduce the operating speeds, and have an undesirable impact on overall operations. There is a direct and inverse trade off between operations and wear in this condition.

Noting the effect of large unbalance values on wheel/rail wear, it was suggested the maximum amount of unbalance be limited to three (3) inches, so as to avoid the large increment in force (and thus wear) that appears to occur for unbalance values greater than 3 inches. However, this was based on wear considerations only, <u>not safety consideration</u>, and as such it operational requirements dictated a higher level of unbalance, this could be accommodated, provided it is within the overall safety envelope of the vehicles. It is this <u>safety</u> criterion that should limit the maximum allowable level of superelevation, as well as elevation.

Critical Measurements and Tolerances

Allowable component tolerances and wear limits can directly effect the wheel and rail wear of the system. For example, in the case of mechanical equipment, any increase in the friction between key truck and car body components can result in increased energy requirements, poor curving performance, and thus increase in wear at the wheel/rail interface.

This latter behavior was specifically observed in the center plate liners (wear plates) used on SEPTA equipment. One design of replacement wear plates was found to have a significantly increased coefficient of friction than the original wear plate. Analysis of the effect of this increased friction wear plate showed a loss in curving performance, which in turn resulted in increased lateral wheel/rail forces and increased energy at the wheel/rail interface. This increase in energy dissipated at the wheel/rail interface translates directly into an increase in both wheel flange and rail gage face wear. Based on this analysis it was recommended that low coefficient of friction center plate liners (wear plates) be used on all SEPTA equipment.

In addition to the above analyses, a parametric study of mechanical tolerances and sensitivities was performed with particular emphasis on key truck clearances, both lateral and longitudinal. Based on this analysis it was found that the gap at the pedestal liner wear plate can effect wheel/rail contact energy. This behavior is illustrated in Figure 4 which shows the effect of these gap dimensions on the contact energy for the AAR-1B wheel profile (the wheel profile recommended in this study). As can be seen from this figure, a very small gap, i.e. less than 3/32", will have a greater level of energy dissipation for a wide range of curvatures. This translates into a higher wheel/rail wear rate. Increasing the gap to a range of between 3/32" and 3/16" for both the lateral and longitudinal gaps results in a reduction of contact energy at the wheel/rail interface. This in turn translates into a lower wheel and rail wear rate.

Rail Wear Standards

Rail wear is one of the key reasons for the removal of rail from track, particularly for transit and commuter track. While there have been significant changes in the criterion used to determine when rail should be replaced in track, rail wear is the dominant replacement criterion for track with light axle loadings, such as transit systems or commuter railways.

Rail wear limits have traditionally been set for jointed track to avoid the flanges of the rolling stock

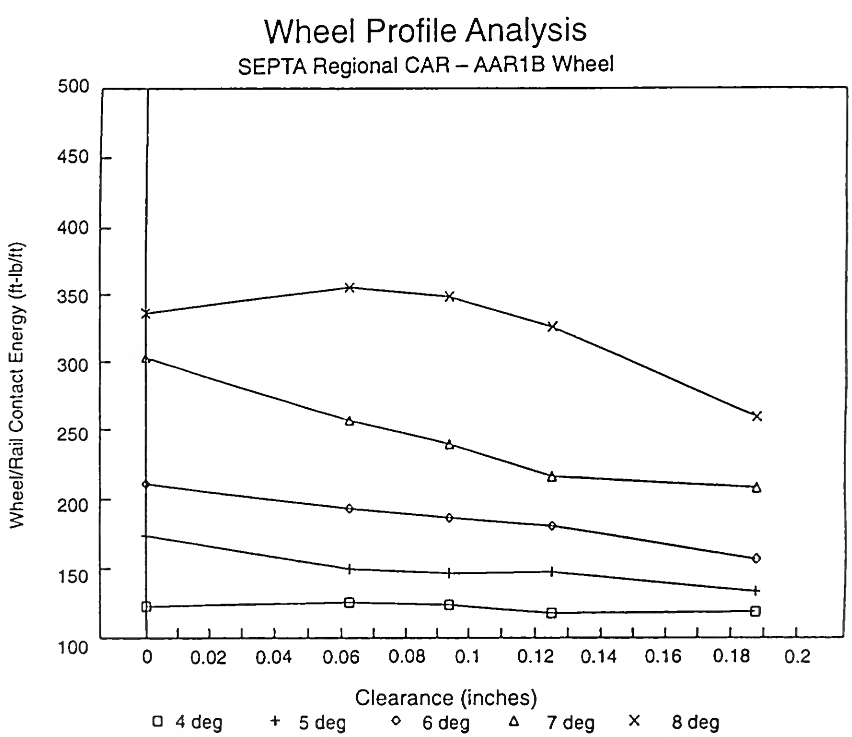

Figure 4. Effect of Pedestal Liners Wear Plate Gap on Wheel/Rail Contact Energy

from striking the top of the joint bars. However, with the use of continuously welded rail and the virtual elimination of rail joints, the bending strength of the worn rail section defines the rail wear limits.

While permissible rail wear limits have been defined to include vertical head loss, side (gauge face) head loss, head area loss, and maximum angle of side wear, only the first two have been used on an active basis in North America because of their ease in measurement and relative simplicity of application. However, interest in the maximum angle of side wear has significantly increased on North American transit and passenger systems because of the risk of wheel climb and associated derailment of equipment. As a result, several North American properties have recently incorporated gage face wear angle limits to their existing rail wear standards.

Vertical head loss refers to the loss of metal from the top surface of the rail head. While it occurs on all track, it is a dominant mode of wear on tangents and shallow curves. Vertical head loss is measured independent of any other parameter, and is taken at the center of the rail head. Gage face wear, or side head loss, refers to the loss of metal from the side of the rail head. Gage face wear occurs primarily on curved track when flanging of the wheels occurs. Gage face wear is usually measured 5/8" below the top of the rail head. Thus, its exact location is dependent on the degree of vertical head wear.

In order to properly define these limits, the strength of the rail, in bending, is calculated using the rail's beam strength and Beam on Elastic Foundation theory. As the rail experiences wear, both head and gage face wear, it's beam strength is reduced, thus increasing the stresses in the rail caused by the

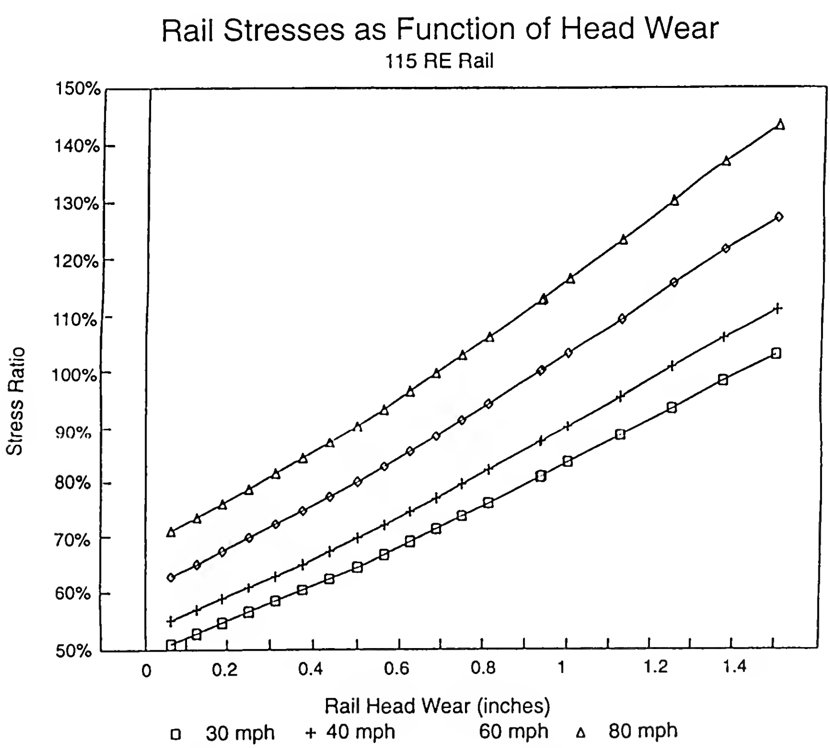

Figure 5. The Effect of Head Wear on Rail Stresses

rolling stock and their associated wheel loadings. Figure 5 presents the results of such an analysis for 115 RE rail showing the loss of rail strength (and thus rail head wear) as a function of maximum operating speed.

This analysis was performed for each of the major rail sections of interest to SEPTA: 115 RE, 100 ARA-B, and 132 RE. The resulting wear limits, both head and gage face wear, were determined for each of these sections and presented in Table 3. In addition, a maximum allowable gage face angle value is also presented based on wheel climb criterion (safety).

TABLE 3

Rail Wear Limits

Rail Section	**Max Head Wear (inches)**	**Max Gage Face Wear (inches)**
100 ARA-B	3/8 - 7/16	3/8 - 7/16
115 RE	5/8	1/2 - 5/8
132 RE	3/4	/8 - 3/4

- Replace if wheel flanges strike top of joint bars
- Maximum allowable angle of side wear 32 degrees from the vertical

Economic Benefit Analysis

As part of this overall activity, cost-benefit analyses were performed for those key maintenance activities where significant additional cost outlays may be experienced by SEPTA. Specifically, the following maintenance areas were analyzed:

- Economics of rail lubrication
- Economics of rail grinding.

These analyses determined the benefit (in reduced component requirements and corresponding costs) of the proposed maintenance activity, together with the costs associated with effectively and adequately implementing these activities.

The primary benefit associated with increased lubrication is a significant reduction in the rate of wear of the rails in track. Additional benefits associated with lubrication include a reduction in wheel wear, which corresponds to the reduced rail wear, and a reduction in fuel (energy) usage. Table 4 presents a detailed breakdown of the savings and costs associated with increased lubrication. These result in a Return on Investment for rail lubrication (ROI) of between 85% and 89%. Note that this does not include any energy savings.

The primary benefit associated with rail grinding is a significant reduction in the rate of rail replacement. The largest such benefit is due to rail profile grinding, while a second benefit is also associated with the reduction and/or elimination of rail surface defects such as corrugations, engine burns, battered welds, etc. Additional benefits associated with grinding include a reduction in wheel wear, which corresponds to the rail wear reduction due to profile grinding. In addition, elimination of corrugations and surface defects will provide benefits in the area of track surfacing and energy (fuel) consumption.

The net savings due to grinding varies significantly based on whether conventional or profile grinding is performed. For conventional grinding, the net savings correspond to an ROI for grinding of approximately 47%. For profile grinding, the net savings correspond to an ROI of approximately 81%. These results are summarized in Table 5.

TABLE 4

Economics of Lubrication
Summary of Costs and Benefits
Rail and Wheel Savings Only

	Moderate Lubrication	"Good" Lubrication
Annual Savings/Cost		
Rail Savings	$ 109,600	$ 171,873
Wheel Savings	$ 153,524	$ 293,091
Total Savings	$ 263,124	$ 464,964
Annual Cost of Lubrication		
Capital Cost	$ 27,333	$ 36,445
Maint Costs	$ 90,090	$ 166,320
Grease Cost	$ 21,622	$ 48,048
Total Cost	$ 139,045	$ 250,813
Net Savings	$ 124,079	$ 214,151
ROI	89%	85%

TABLE 5

Economics of Grinding
Summary of Costs and Benefits

	Defect Grinding	Profile Grinding
Annual Savings/Cost		
Rail Savings	$ 113,153	$ 169,730
Wheel Savings	$ 63,216	$ 153,524
Surfacing	$ 2,066	$ 4,806
Energy Savings	$ 2,776	$ 6,310
Total Savings	$ 181,211	$ 334,369
Cost of Grinding	$ 123,000	$ 184,500
Net Savings	$ 58,211	$ 149,869
Return on Investment	47%	81%

Conclusions

The wheel and rail wear that was being experienced by SEPTA's Regional Railroad Division was predominately wheel flange/rail gage face wear, particularly on moderate to sharp curves on the commuter rail lines.

In order to analyze the wheel/rail dynamics associated with SEPTA's operations and to evaluate the effects of changes in maintenance practices on wheel and rail wear, a vehicle-track curving computer model was used to analyze the curving behavior of a complete SEPTA rail vehicle. Based on this model analysis, together with the analysis of the wheel and rail behavior itself, a serious of recommendations for changes in maintenance practices, policies, and standards were developed.

Among the recommended areas of change were the use of improved rail and wheel profiles. In the case of the rail profiles, use of an asymmetrical rail head profile, obtained through rail profile grinding, was suggested as a means of controlling rail wear, and in particular rail gage face wear. Specifically, a profile that generates and maintains "one point contact" between the flange of the wheel and the gage corner of the rail was developed for the sharper curves, while alternate (different) profiles were developed for tangent track and the high and low rails on shallow curves (3 degrees or less).

In addition to the control of the rail profile, rail grinding was also recommended for the control of corrugations, battered welds, engine burns, and other rail surface defects.

For the wheel profiles, SEPTA's standard AAR 1:20 profiles was compared to the newly developed AAR 1B profile. Based on model analysis, it was determined that the AAR-1B profile improves the effective conicity of wheel, thus improving steering of wheelset, and as a result, the AAR-1B reduces flanging, and thus wear, on moderate curves. Based on this analysis, the AAR-1B profile was recommended for control of wheel-rail wear on SEPTA.

Another area of emphasis that emerged from this study was that of proper lubrication. It was ascertained that more "effective" rail lubrication practices can extend rail wear life to two or more times that of current curve rail life, in many cases. In order to achieve this, it was recommended that SEPTA investigate and implement a hi-rail mounted system for lubricating its curve or a more comprehensive wayside lubricator system. Such a hi-rail vehicle could apply a layer of lubricant, accurately to the gage face of the rail, continuously along the curves. Alternately, a more extensive wayside lubrication system would be required. However, the effectiveness of these wayside lubricators must be increased significantly by placing a strong emphasis on lubricator maintenance.

While wheel/rail lateral forces, and thus wheel/rail wear, will increase as a function of increasing overbalance or increasing superelevation, unbalance permits the operation of higher speeds on track, within a defined safety envelope. Thus, reducing allowable unbalance will directly reduce the level of safe operating speeds, and have an undesirable impact on overall operations. There is a direct and inverse trade off between operations and wear in this condition. Noting that there are other ways to effectively control wheel/rail wear, in the case of SEPTA operations, other techniques such as increased lubrication and grinding represent a higher priority for the control of wheel and rail wear.

Mechanical equipment tolerances and wear between key truck components can likewise affect wheel/rail wear, as can an increase in the friction between key truck and car body components. This latter behavior was specifically observed in the center plate liners (wear plates) used on SEPTA equipment. Analysis of the effect of this increased friction wear plate showed a loss in curving performance, which resulted in increased lateral wheel/rail forces and thus an increase in both wheel flange and rail gage face wear. Use of low coefficient of friction center plate liners (wear plates) would significantly reduce this effect.

Rail wear limits were also redefined; to include: vertical head loss, side (gauge face) head loss, and maximum angle of side wear. Making use of the strength of the rail, in bending, allowable limits for both head and gage face wear were calculated for each of the major rail sections of interest to SEPTA: 115

RE; 100 ARA-B; and 132 RE. In addition, a maximum allowable gage face angle value was established based on wheel climb criterion (safety).

Finally, in order to determine whether many of these "improvements" were in fact economically justifiable, a cost-benefit analyses was performed, with specifically emphasis on the economics of rail lubrication and the economics of rail grinding.

The resulting analyses showed a Return on Investment (ROI) for rail lubrication of between 85% and 89% and an ROI for rail grinding of between 47% and 81%. Thus, the recommendations for reducing rail and wheel wear appear to be economically justifiable as well as technically feasible.

Recent application by SEPTA of several of these recommendations has resulted in a noticeable decrease in the rate of wheel and/or rail wear, and it is expected that with the full incorporation of these recommended changes in practice, wheel and rail life will experience further improvements.

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Association of American Railroads
Research and Test Department

RAIL LONGITUDINAL FORCE MEASUREMENT EVALUATION STUDIES USING THE TRACK LOADING VEHICLE

Report No. R-834

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May 1993

Chicago Technical Center
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1.0 INTRODUCTION

While the increased use of continuous welded rail (CWR) in North America has substantially improved rail performance, it has also led to an increase in track buckling incidents. In an effort to improve the safety and performance of CWR tracks, experimental and analytic investigations were conducted by the Volpe National Transportation Systems Center (VNTSC) supporting the research mission of the Federal Railroad Administration (FRA). These investigations resulted in the development and validation of dynamic buckling analyses models and buckling safety criteria [1]*, and the development of measurement techniques for the determinations of track lateral resistance and rail longitudinal force [2].

The knowledge of rail longitudinal force is extremely important for buckling prevention for it provides a means to determine the rail's neutral temperature, and hence enables the direct application of buckling safety criteria and buckling prevention techniques. As has been shown in previous works [3] [4], rail neutral temperature is a highly variable quantity and generally is not the same as the rail's installation temperature.

The neutral temperature is defined as the temperature at which the net longitudinal force in the rail is zero, and as shown in [3], neutral temperature can be related to by the equation:

$$P = AE\alpha(T_{rail}-T_N)$$

where, **P** is the longitudinal rail force, **A** is the rail cross-sectional area, α is the coefficient of thermal expansion, $\mathbf{T_{rail}}$ is rail temperature at the time of measurement and $\mathbf{T_N}$ is the rail neutral temperature. Hence, once **P** is known, $\mathbf{T_N}$ is also known.

Feasibility of a rail longitudinal force measurement technique rests on the ability of the technique to; (1) measure rail longitudinal force within ±12.5 kips (±5 °F neutral temperature change), (2) measure absolute rail force and be non (or minimally) destructive to the rail tie structure (3) be impervious to rail microstructure, residual stress influences and track variations, and (4) be easily field deployable with

* Numbers in brackets refer to References listed in Section 8.0

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RAIL ROAD
CROSSING
YIELD
UNION PACIFIC
STOP
R R
LOOK OUT FOR THE CARS
YARD LIMIT

real-time output of **P** and $\mathbf{T}_N$. Research at VNTSC has focused on the development of a measurement approach based on beam-column theory. Proof of concept was demonstrated through a four year test program conducted in conjunction with the Association of American Railroads and Foster-Miller, Inc (FMI) at the Transportation Test Center in Pueblo, Colorado which focused on the development of governing theoretical beam equations and subsequent concept implementation and testing with a car-mounted uplift system [2]. These tests confirmed the rail uplift concept's viability, and established accuracy limits and measurement feasibility. Recently, the technique has advanced to a stage of dedicated car deployment and subsequent revenue service applications. This has been accomplished through a joint effort under the FRA/VNTSC Track Safety Research Program and the AAR's Vehicle Track System (VTS) Program.

The primary objective of the VTS program was to bring about a systems view method to analyze vehicle and track interaction problems, to reduce track and equipment costs, and to improve the safety of train operations. Construction and demonstration testing of the Track Loading Vehicle, the quantification of the lateral strength characteristics of in-place railroad track, testing and validation of various derailment criteria, and bridge tests were among the previous activities performed under this project [5,6]. In conjunction with this research program, a new interchangeable load bogie was designed and constructed at the Chicago Technical Center for use in rail uplift tests. The testing of the TLV described in this report was supported by the FRA under the auspices of the Track Train Interaction Derailment Analysis Project under Sub-task 6d of Task Order 6 of Contract DTFR53-86-C-00011. The purpose of this report is to provide an overview of the rail uplift concept, to present the results of a feasibility/calibration study performed with the TLV, and to present results of revenue service rail force/ neutral temperature measurement studies on a major Western railroad employing the TLV.

2.0 GOVERNING THEORY: BEAM-COLUMN RESPONSE APPROACH

The basic principle of the beam-column approach, as demonstrated in Exhibit 1, is that the longitudinal force level in the rail will measurably affect rail deflection under an applied load. In practice, the load can be applied either vertically or laterally. Both methods have their advantages; however, vertical loading lends itself to an easier car-mounted approach and thus for an easier field deployment.

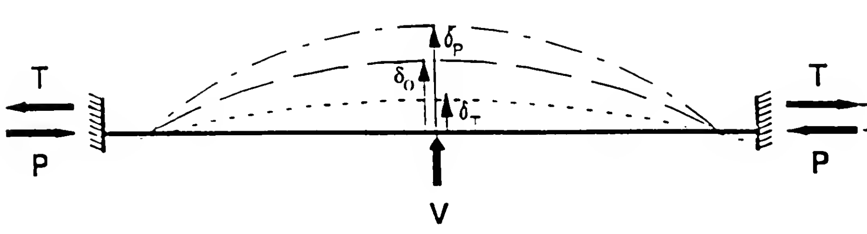


Exhibit 1: Basic Principle Illustrating the Beam Column Approach for Rail Longitudinal Force Measurement.

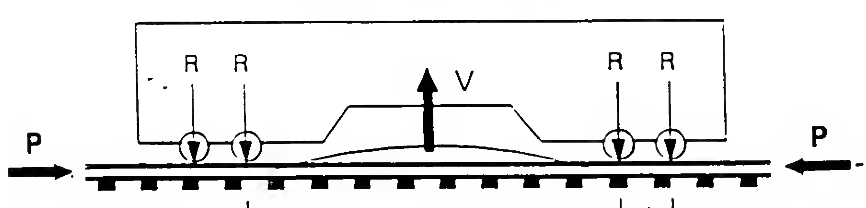
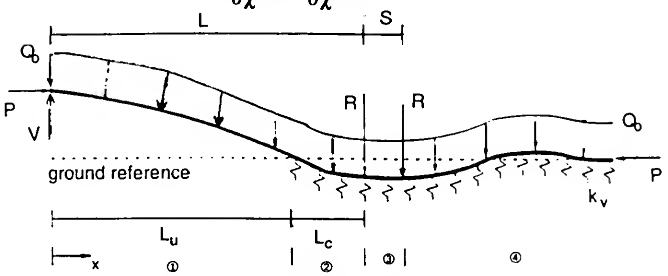


Exhibit 2: Car Mounted Configuration.

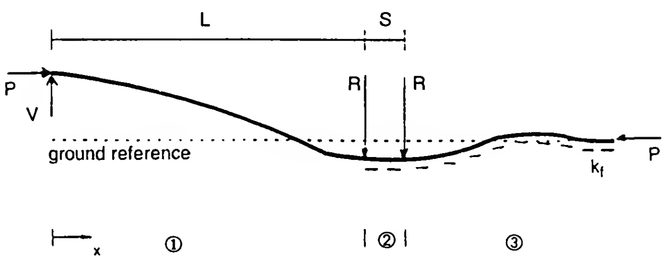
An analytic model of the vertical deflection response of a straight rail to an applied vertical load was developed to estimate measurement accuracy and sensitivity and to predict the required uplift forces at specific deflection levels. The analysis models a car-mounted uplift configuration (Exhibit 2). It consists of an infinite beam with constant modulus of elasticity, **E**, cross-sectional area, **A**, and moment of inertia, **I**, governed by a set of vertical equilibrium equations:

$$EI\frac{\partial^4 w}{\partial \chi^4} + P\frac{\partial^2 w}{\partial \chi^2} + k_v w = Q_o \tag{1}$$



where, Q_o = rail self weight, R = wheel load, V = applied vertical load, S = axle spacing, and 2L+S = truck center spacing, and a set of longitudinal equilibrium equations given by:

$$AE\frac{\partial}{\partial \chi}\left(\frac{\partial u}{\partial \chi} + \frac{1}{2}\left(\frac{\partial w}{\partial \chi}\right)^2\right) = k_f u \tag{2}$$



The model specifically accounts for the influence of track foundation modulus, k_v, and track longitudinal stiffness, k_f, as well as variation in rail size and car-type. All charts and/or graphs representing theoretical analysis presented within this report, will be referenced to parameters dictated by the TLV, namely truck center spacing = 561", axle spacing = 108" and wheel load = 33 kips.

An important feature of this approach to rail longitudinal force measurement is the linear relationship between the longitudinal force P, and the vertical uplift force V. Exhibit 3 illustrates the linear P-V relationship and shows the influence of rail cross-sectional properties, specifically, A and I on the response. *(Note: the slope of the curve, dV/dP , is the "**sensitivity**" of the P-V response characteristic.)* Curvature influences are slightly harder to represent in a similar concise manner. They encompass influences due to an extension/reduction in uplift rail length from a tangent scenario (see Exhibit 13), and end conditions influences due to a change in vertical wheel loads due to superelevation. These effects will become clearer when observing the 5° test results from the TLV feasibility studies in Section 4.

Curved beam bending-torsional influences in the vertical plane are neglected in the analyses due to typically very large radii of curvatures. For detailed results of rail uplift feasibility evaluation tests and concept verification studies, refer to [2].

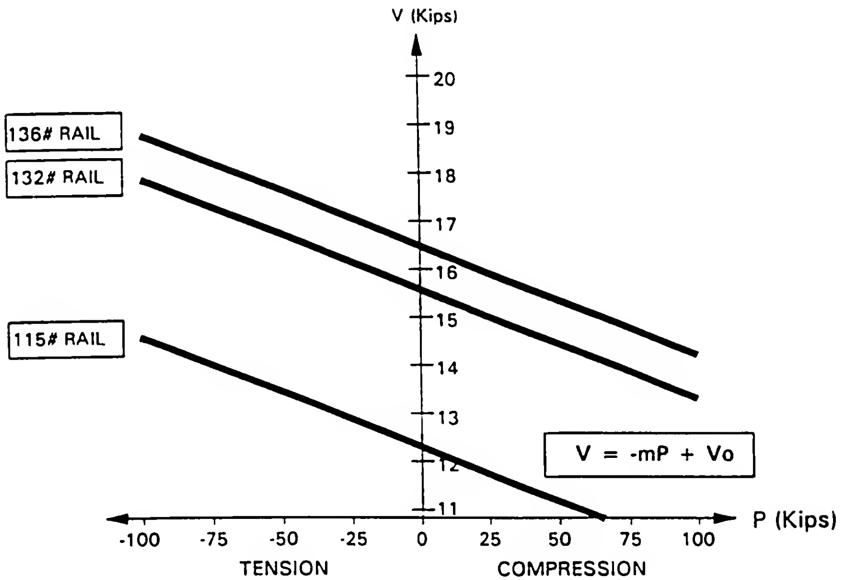


Exhibit 3: Linear P-V Relationship.

3.0 DESCRIPTION OF THE TLV RAIL UPLIFT SYSTEM

The Association of American Railroads has designed, constructed, and successfully tested a Track Loading Vehicle (TLV) which is the most advanced research vehicle of its kind in the world. The test vehicle is operated from an instrumentation car and equipped with two locomotive trucks with a special load bogie mounted underneath the center for applying computer controlled loads to measure the response of the track. Built on the underframe of a 4,000-hp diesel-electric locomotive, the 263,000-pound TLV is used to apply forces close to the strength limits of the rails and other track-structure components such a ties, rail fasteners, and ballast. At the heart of the vehicle is an electro-hydraulic control and a computer system which provide the high-speed controls vital to conducting tests without damaging track. The loads are applied to the track through a center "load" bogie using servo-controlled hydraulic actuators. Exhibit 4 shows a photo of the Track Loading Vehicle and the AAR 100 Instrumentation Car. In conjunction with a joint AAR/FRA Vehicle Track Systems Project, the AAR has designed and constructed a new interchangeable rail uplift bogie (Exhibit 5) for use in rail longitudinal force measurement on revenue trackage. The rail uplift bogie is a box structure which is mainly constructed with structural I-beams bolted together at the four corners. Two other I-beams are placed on to the box structure transverse to the track and carry a pair of slides on linear bearings.

The hydraulic equipment for the TLV rail uplift loading system consists of six independently controlled servo-hydraulic actuators. Two feedback controlled vertical bogie actuator systems, rated at 55-kip each, are used in displacement control mode to lower and raise the load bogie. Two lateral actuators, rated at 22 kips each, are used to center the bogie over curves in conjunction with a bell-crank mechanisms. These actuators are also used to keep the bogie parallel to the track during lift. Rail lift is achieved by two additional 22-kip vertical lift actuators attached to the movable slides which ride on the transverse beams in Exhibit 6. The upper connection of the lift actuators can pivot to enable the actuators to self align perpendicular to the load frame when load is applied.

Exhibit 4: Track Loading Vehicle.

A ground reference bar, shown in Exhibit 7, is used for mounting displacement transducers in order to measure the rail lift height. A pair of ground stakes to which the Ground Reference Bar (GRB) is mounted are driven into the subgrade. The stakes are placed outside the tie area to minimize the possibility of motion of the reference system during the lift. The GRB itself is a 2 x 2 inch square aluminum tube, 10 feet long. It is referenced by resting it across the track on top of two 4 inch blocks (to accommodate rail clearance for a maximum of 3-inch lift) that sit on top of the rail heads. This insures that the GRB is parallel to the rails, regardless of the superelevation, and high enough above them to clear the rail(s) at the final lift height. The initial position of the load bogie with respect to the GRB is established by using a pair of digital levels placed on the bogie and the GRB. The

Exhibit 5. TLV Rail Uplift Bogie.

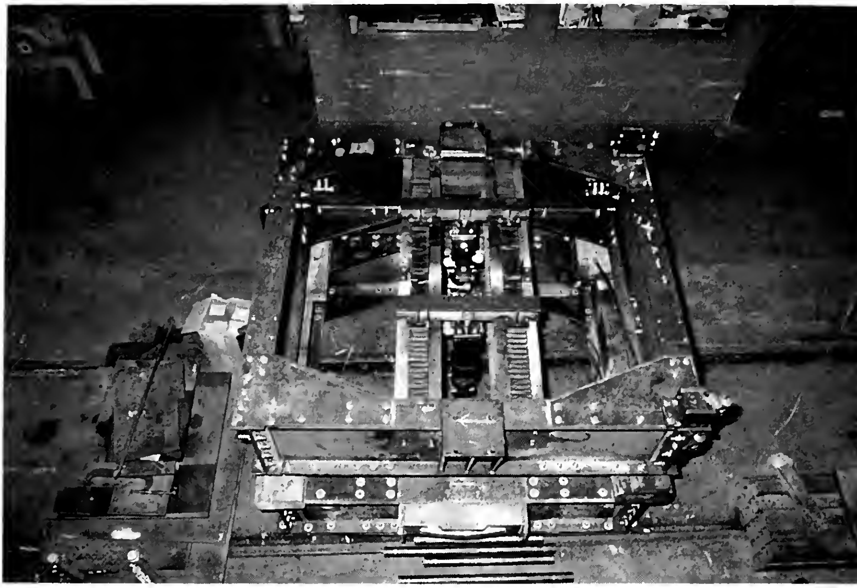


Exhibit 6: Rail Uplift Bogie Lift Mechanisms, Top View.

vertical bogie actuators are adjusted up and down until the digital level on the load bogie reads the same cross level as the one placed on the GRB.

A five-piece hook mechanism is attached to the end of each lift actuator to clamp the rail head. Once the lift actuator is centered over the rail, it is lowered by hydraulic controllers and the hook mechanism is attached securely to each rail head. Four displacement transducer brackets are mounted on the GRB for measurement of lift and bogie vertical displacements. Measurement of the lift height is taken directly from the displacement transducers mounted on the ground reference frame to the top of the rail head. Measurement of the lift load is taken directly from the MTS load (DC) controllers mounted on the lift actuators. The absolute motion of the rail head to the ground is measured by a pair of rail head to GRB displacement transducers. On either side of the load frame, as near to the center of the lift point as possible, is a ground reference LVDT which measures the relative displacement between the load bogie and the ground reference frame. The resulting measurements are used in an outer feedback control loop designed to maintain the load frame position both parallel to the track and at a fixed height above it. During the lift, this feedback loop actively adjusts the vertical actuators to maintain the load frame position with respect to the ground reference bar. By this method, any variation in position of the load frame that may be caused by car body motion is compensated for. Compression of springs on the running gear reacting against the lift force and wind induced car body roll are two such causes of motion for which the computer system provides compensation. Exhibit 7 shows the rail hooks, and the associated displacement transducers.

Exhibit 7. TLV Rail Uplift Ground Reference Frame and LVDTs.

4.0 TLV/RAIL UPLIFT - LONGITUDINAL FORCE MEASUREMENT FEASIBILITY STUDIES

Feasibility tests were performed on the TLV uplift system to evaluate the above design modifications and suitability for both tangent and curved track applications, to establish the system's accuracy, and sensitivity limits, and to determine calibration requirements for revenue service deployability.

Inherent in the uplift approach is a trade-off between sensitivity and accuracy. Past an optimum point, sensitivity will increase as accuracy begins to decrease. Sensitivity is the extent to which the device is able to discriminate between differing longitudinal force levels. It is expressed by the slope of the P-V curve in kips of uplift force, V, per kips of longitudinal force, P. Accuracy is the extent to which the device measures the "true" longitudinal force level in the rail. To demonstrate these characteristics, the above analytical model was used to calculate error associated with not knowing k_v and k_f for each measurement run. As discussed earlier, the uplift device, to be practical, must be capable of measuring longitudinal force to within ±12.5 kips, for a range of forces from -100 kips (tension) to +100 kips (compression). The error in longitudinal force measurement due to k_v (2000-5000 psi) variation was calculated to be within ±4.6 kips and due to k_f (100-300 psi) variation to within ±0.5 kips at a 2.0" uplift deflection. This shows that k_v and k_f variation have a small influence on the measurement.

One parameter, however, that does play a critical role in both the sensitivity and the accuracy of the system's P-V response is the *"exactness"* of the uplift deflection measurement. Slight errors in this measurement, (seen as δ in Exhibit 1), will greatly influence the error in reading the *"true"* longitudinal force. Exhibit 8 replicating an actual test measurement exemplifies the criticality of this issue. As can be seen, at a 2.0" deflection level, a 0.1 inch error in the uplift measurement could lead to a 1 kip uplift force difference.

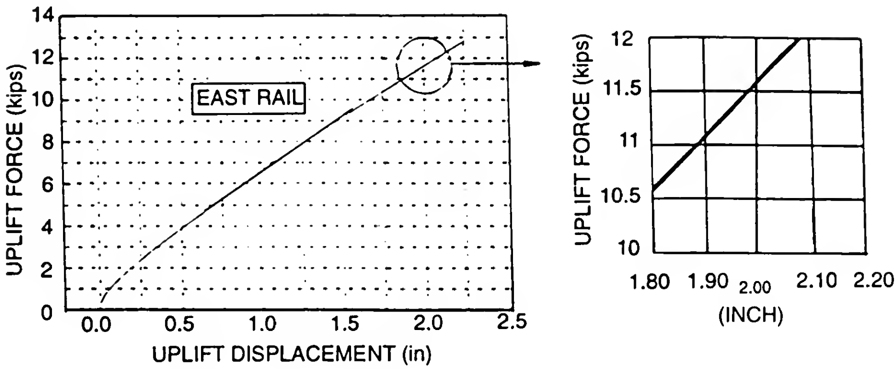


Exhibit 8: Criticality of System Uplift Measurement Accuracy.

Calibration and feasibility studies encompassed nine field test zones, three on tangent track and six on five degree curved segments at the Transportation Test Center. The basic test approach was to determine the rail longitudinal versus uplift load characteristics, the P-V response of the system, at a two inch rail uplift deflection by varying thermally induced force levels. Key measurements recorded were the rail uplift force and deflection, rail longitudinal force, wheel loads, and rail profiles. The TLV had the capability of lifting both rails simultaneously or each rail one at a time. On tangent track, no distinguishable variation was observed between these respective pull techniques, however, some variation did occur on the five degree test site. While these variations were slight, they did have an effect on the sensitivity of the P-V response, by affecting the slope of the resulting P-V curve. It is believed, that the shift in sensitivity is related to a differing uplift load sharing at the wheels due to load path differences for single vs dual uplift in presence of superelevation. The change in load path essentially alters the end conditions posed in the current test section. Exhibit 9 attempts to graphically depict this relationship while Exhibit 10 shows the change in P-V responses experienced within the 5 degree test results.

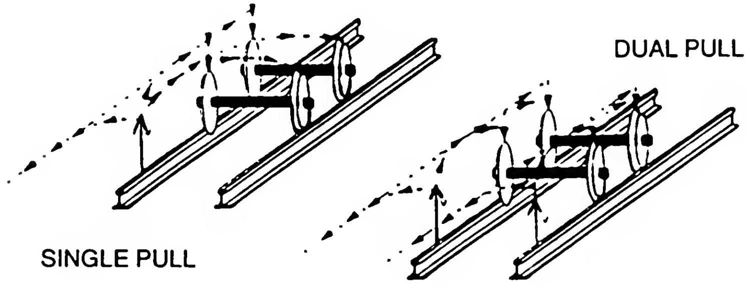


Exhibit 9: Single Versus Dual Load Path.

Final results for the tangent test section and the dual pull 5° curve sections are summarized in Exhibits 11 and 12. From here on, the curved results will be referenced to dual pull testing only, since a dual pull test was decided to be the most practical approach to subsequent revenue service testing.

As can be seen, excellent results were obtained from all tangent test sections with 100% of the data falling within the ±12.5 kips accuracy range as specified in the test requirements. Slightly more scatter was observed among the five degree test results with 90% of the data falling within the required accuracy limit. It is conjectured that the scatter to some extent was the result of rail moving out laterally during the pull, and also due to vertical residual rail deflections during multiple lifts at the same location.

Several factors influence the P-V response for curved track. They are a change in subtended rail length due to the car body resting on curved track and an unequal distribution in wheel loads due to superelevation effects. These factors together result in the separation in regression lines between high and low rail as schematically indicated in Exhibit 13.

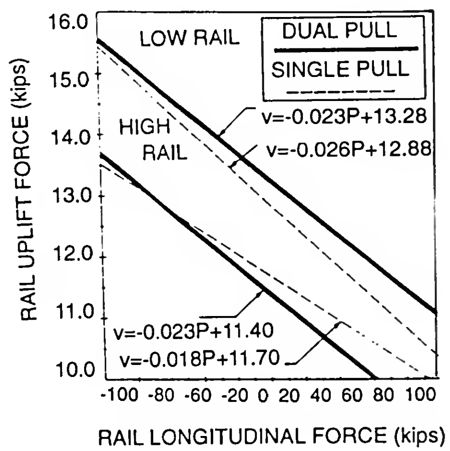
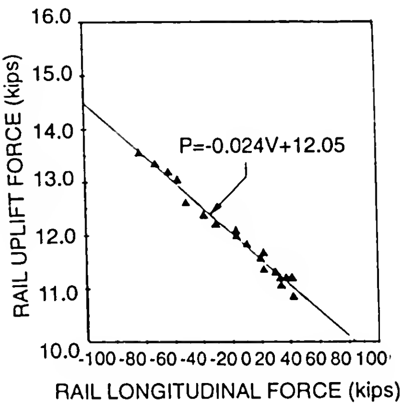


Exhibit 10. Shift in the P-V Response of a 5° Curve by Comparing Dual Pull Test Sections to Single Pull Test Sections.



Exhbit 11. Tangent Test Results

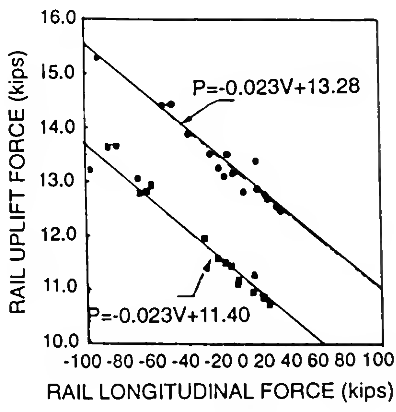


Exhibit 12: 5° Test Results

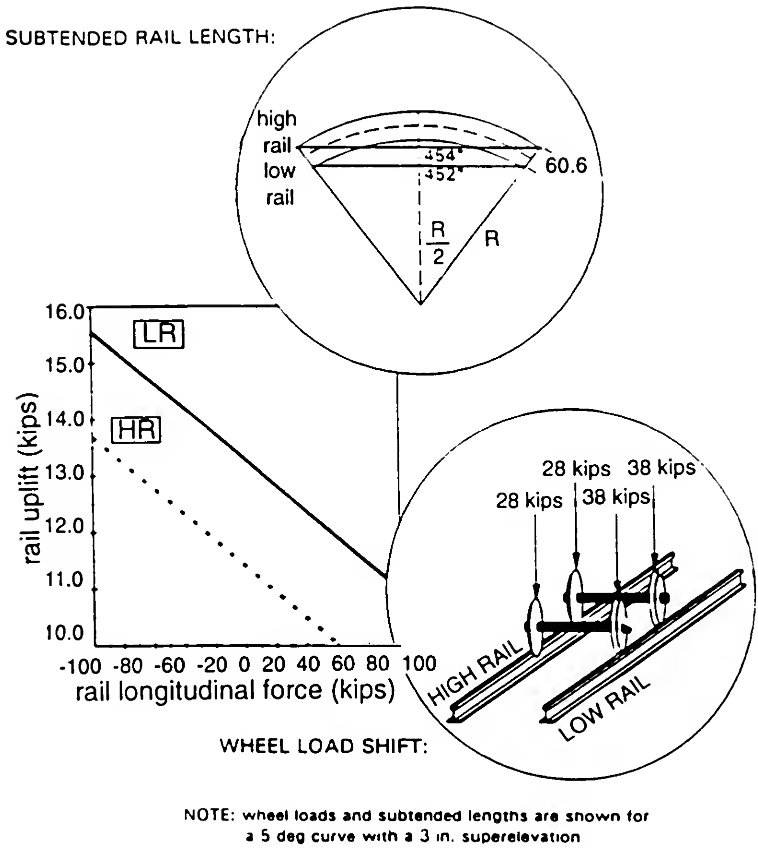


Exhibit 13: The Effect of Curvature Influences on the P-V Response of Curved Track.

5.0 RAIL LONGITUDINAL FORCE PREDICTOR MODEL

Overall, the TLV rail uplift measurement system proved capable in obtaining very good measurement accuracy and sensitivity. By coupling the rail uplift pilot test results with mechanics theory, the development of a generalized "rail longitudinal force predictor model" was possible. The *"predictor"* was needed for TLV revenue service applications. It provided a means of real-time display for both rail longitudinal force and rail neutral temperature for a variety of track conditions, *i.e.* rail size, curvatures, and superelevations.

The following field variables were identified in the development of an ***"universal"*** predictor model for the determination of rail longitudinal force using the previously described Rail Uplift Test (RUT):

- Rail size and flexural rigidity
- Curvature influence on subtended beam length, L
- Wheel load influence on boundary conditions, determined by superelevation
- Foundation modulus, k_v, influence on the initial rail profile (causing non-uniform reference frame for uplift measurement)

The last issue above, should ideally be solved by a vehicle mounted uplift measurement, referenced to the wheels. Alternatively, the initial deflections or vertical rail profile can be measured using a transit at both inner wheels and at the pull point. For the purpose of imminent revenue service applications, variation in k_v and initial profile is considered negligible, due to the majority of the test locations being on good, stiff track conditions.

The development of an ***"universal"*** predictor model for revenue service TLV applications, consists of utilizing the predictions of a model based on Equations (1) and (2) for tangent tracks, and developing appropriate modification factors for curved tracks to account for the superelevation induced wheel load change influences, and the subtended rail uplift length differences. These modification factors are determined based on fitting theoretical results against the 5 degree TLV high and low rail test data, together with an empirical relationship (based on analysis and tests) relating TLV wheel loads to measured superelevation. The resulting predictor model then requires as an input a minimum number of easily measurable parameters, and the uplift force at 2.0 in deflection to provide a real time output of rail force and rail neutral temperature as schematically shown on Exhibit 14.

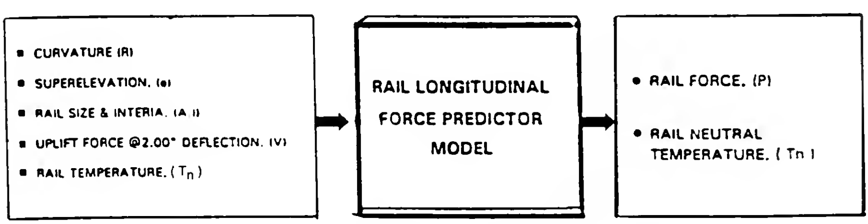


Exhibit 14: Rail Longitudinal Force Predictor Model.

6.0 REVENUE SERVICE RAIL FORCE MEASUREMENTS

The primary objective of these tests was to characterize CWR rail force and neutral temperature variation on ***"typical"*** and ***"buckling sensitive"*** revenue service track. The secondary objectives were to evaluate the TLV car mounted rail force measurement system, and its adaptability to revenue service type applications, and to simplify the technique to "more routine" diagnostics applications. This information in turn could be used to prevent buckling or pull-aparts. This section summarizes the findings and results.

The track to be tested was surveyed a week prior to the TLV arriving on site. The superelevation, curvature and rail section profile were noted. The profile data obtained from these readings were reduced to give a moment of inertia and across sectional area for the test locations.

The general test procedure was as follows. The test sites were prepared in advance of the TLV arriving on site. The fasteners, either clips or cut spikes, were removed on either side of the pull point on both rails to a distance equal to the TLV inner wheel spacing. The TLV was then spotted and rail guides were installed on the bogie (through the curved test sections) to avoid any lateral movement that might occur. The uplift clamps were then placed on the rail, and the bogie was raised at a rate of 100 pounds per second. This continued until the rail was lifted 2.25 inches, and held at this point for a few seconds. The rail was then lowered back into its original position, and the track work- forces reinstalled the fasteners. The TLV was then moved to the next site, and the procedure was repeated. A post processing program was used to calculate the exact uplift lift force at a deflection of 2 inches. Given this information, along with the rail temperature and cross-sectional area, it was possible to calculate the longitudinal force and the neutral temperature of the rail at that point. An illustration of the rail uplift bogie, along with the accompanying wayside instrumentation and braces is shown in Exhibit 15. Exhibit 16 shows rail being lifted on tangent track, without the use of braces.

Exhibit 15: Rail Uplift Bogie.

6.1 TEST SECTIONS

The test sites were chosen as they had historically been a problem area with regards to derailments and track alignment. Three sections were chosen; Section 1 had concrete ties with Safelok fasteners. The concrete ties were installed and the rail was de-stressed in May of 1992. The low rail was new 136#, while the high rail was 10 year old 132#. A grinding train was through in 1992. Section 2 had wood ties and cut spikes, and had a major tie program in 1984. Sites were chosen on tangent track, and in two curves. The rail was 132# on both the high and low rail for tangent track and the first curve. For the second curve, the low rail was 136#. It was also ground in 1992. Section 3 had concrete ties, also with Safelok fasteners. The ties were installed in 1988, at the same time that the area was surfaced and undercut. The rail was a mixture of 132# and 136#, and was ground in 1992. All test sections have varying grades and curvatures. At some locations, the head wear was very close to the rail's replacement limit. Simplified track charts of the three locations are shown in Exhibit 17 through 19. The tonnage on this line is roughly 47 MGTs, being mostly unit coal trains, with loaded cars heading south. The test locations were 300 or 500 feet apart, depending on the test section, in order to get an idea of the variation in neutral temperature over distance. Additional test sites were chosen near bridge abutments and switches, areas which tend to experience longitudinal force build-up. Exhibit 20 and 21 show locations typical of those tested.

Exhibit 16. Rail Being Lifted.

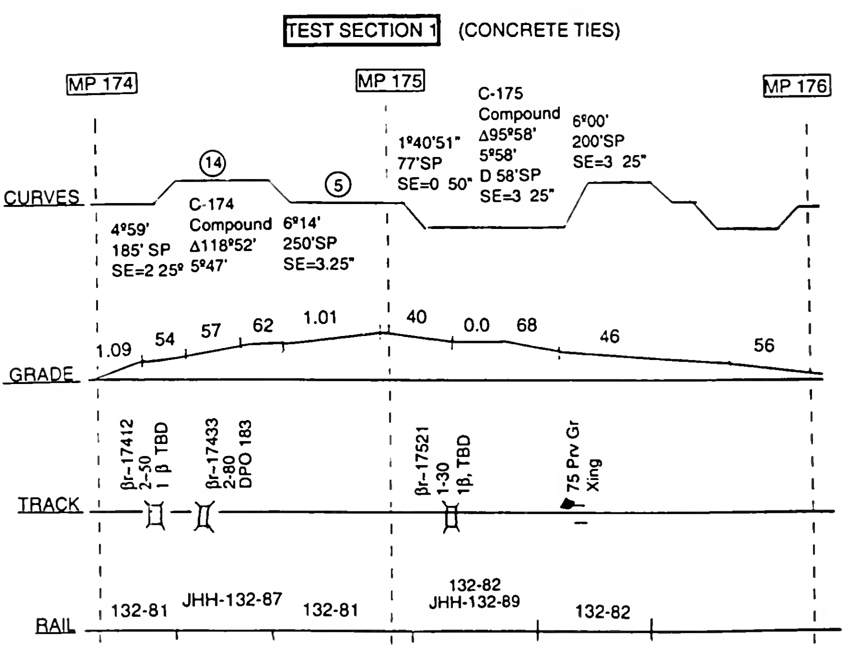


Exhibit 17. Test Section 1.

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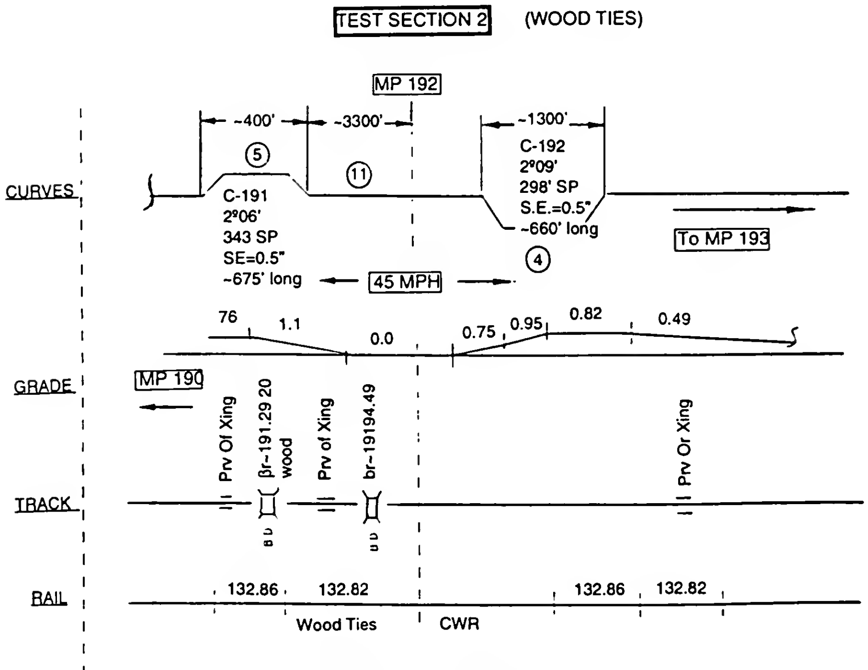

Exhibit 18. Test Section 2.

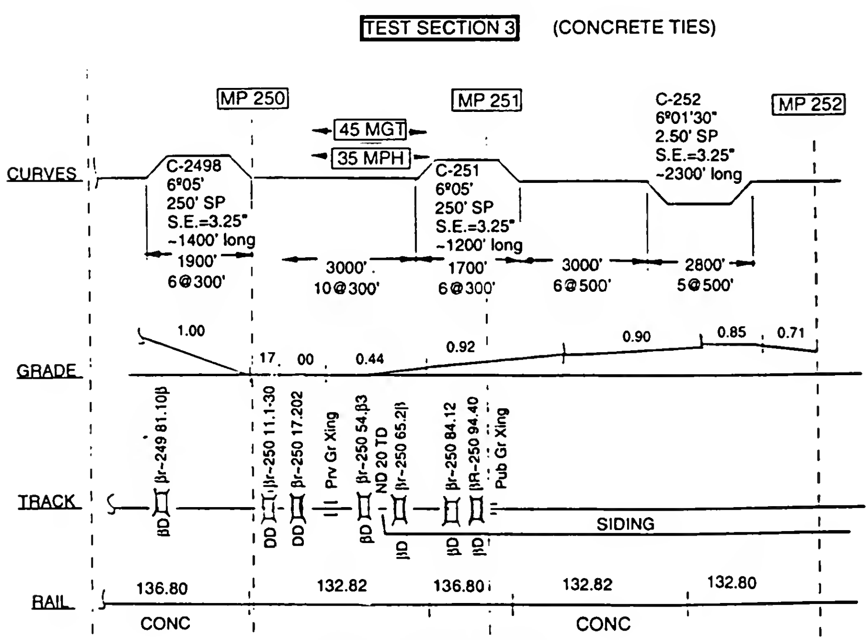

Exhibit 19. Test Section 3.

Exhibit 20. Test Location.

Exhibit 21. Test Location.

6.2 RESULTS

Exhibits 22 through 24 illustrate how the neutral temperature varied over the three test sections. The two rails acted independently of each other in terms of their neutral temperatures, with often significant differences between the two. There were also occasionally significant differences in neutral temperature between two adjacent test locations. An example of this is in section 3, between locations 1 and 2. There were many field welds in this area, so the neutral temperature is generally expected to vary depending on the specific welding temperatures and conditions and the extent to which the rail was subsequently de-stressed or readjusted.

Another example of this variation is in Section 1, where the neutral temperature gradually decreases from location 1 through location 9, then suddenly, at location 10, it jumps back up again. This may be due partly to the presence of many field welds as well as due to a combination of transitioning from curve to tangent through a spiral and perhaps a "dip" in the track. As was described earlier in this report, a large difference between the actual rail temperature and its neutral temperature results in a significant build-up of longitudinal stresses. Should the neutral temperature be too low, the compressive forces present in the rail may result in lateral misalignment such as sun kinks. During the tests, under controlled conditions, some rail lateral movement did take place where there may have been a large amount of longitudinal forces present. Exhibit 25 illustrates this movement, and the importance of the track structure in restraining these forces.

Bridge locations did not show a marked difference from the rest of the area. In some instances the neutral temperature increased, in others it decreased. It should be noted that stress distributions at or near bridges depend on the type of bridge, as well as the track construction leading up to it, such as tie type and anchoring pattern.

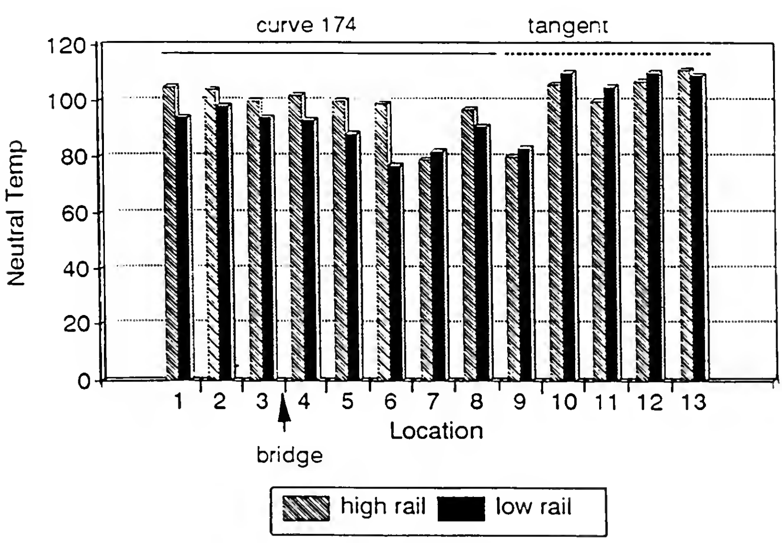

Exhibit 22. Neutral Temperature Variation for Section 1.

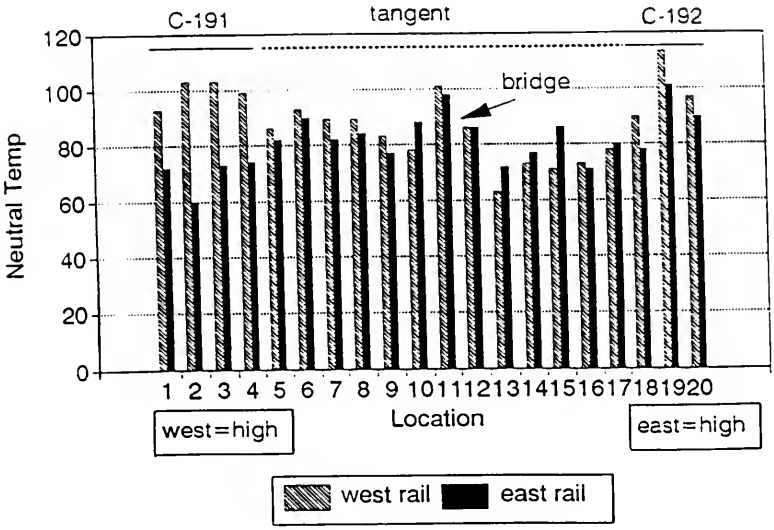

Exhibit 23. Neutral Temperature for Section 2.

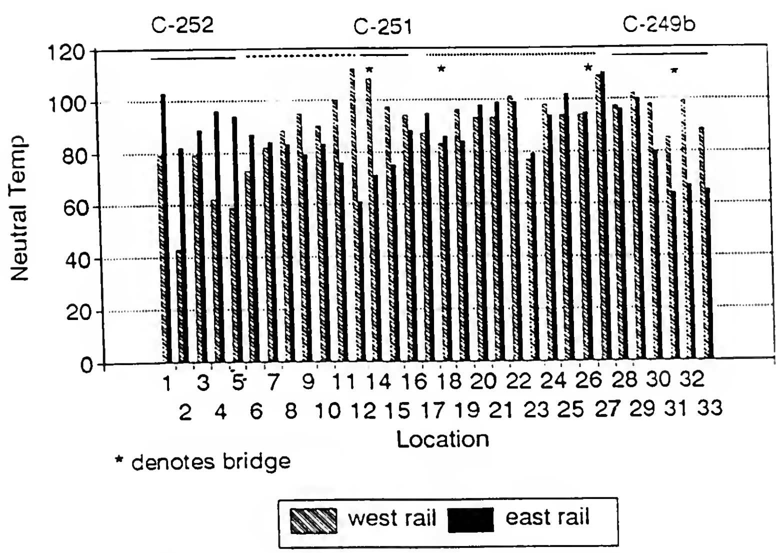

Exhibit 24. Neutral Temperature Variation for Section 3.

The graph for section 2 shows a large difference between the neutral temperature for the high and low rails. This could potentially cause concern at certain temperatures on curves if one rail is in compression and pushes out, while the other rail is in tension and pulls in, causing an increase in gage. For this reason, care should be taken to lay both rails at roughly the same temperature. As an example of a possible buckling sensitive location, 3-02 with an average neutral temperature of only 63° F could present a problem for a 6° curve, especially with weakened ballast resistance and lateral alignments.

The moment of inertia calculations are crucial to the accuracy of the uplift calculations. A lighter rail takes less force to lift, as can be seen in Exhibit 3. As an example, a new 132# RE rail has a cross-sectional area of 12.95 in^2 and a moment of

Exhibit 25. Example of Rail Movement.

inertia of 94.9 in^4. During the testing in section 3, there were several locations that had substantial head wear. The high and low rail profiles of one such location are shown in Exhibit 26. A worn profile, depending on the condition and wear pattern, can have a large effect on the moment of inertia and then on the calculated longitudinal forces in the rail. It was expected that high forces would be apparent at the bottom of grades, and the inverse for crests, however, this did not seem to be the case.

Location 2-02 was one location where the neutral temperature was quite low, 60 degrees. The railroad was concerned about this location and cut the rail. 1 9/16 inches of rail were removed, and the track was rewelded at a nominal temperature of 95 degrees.

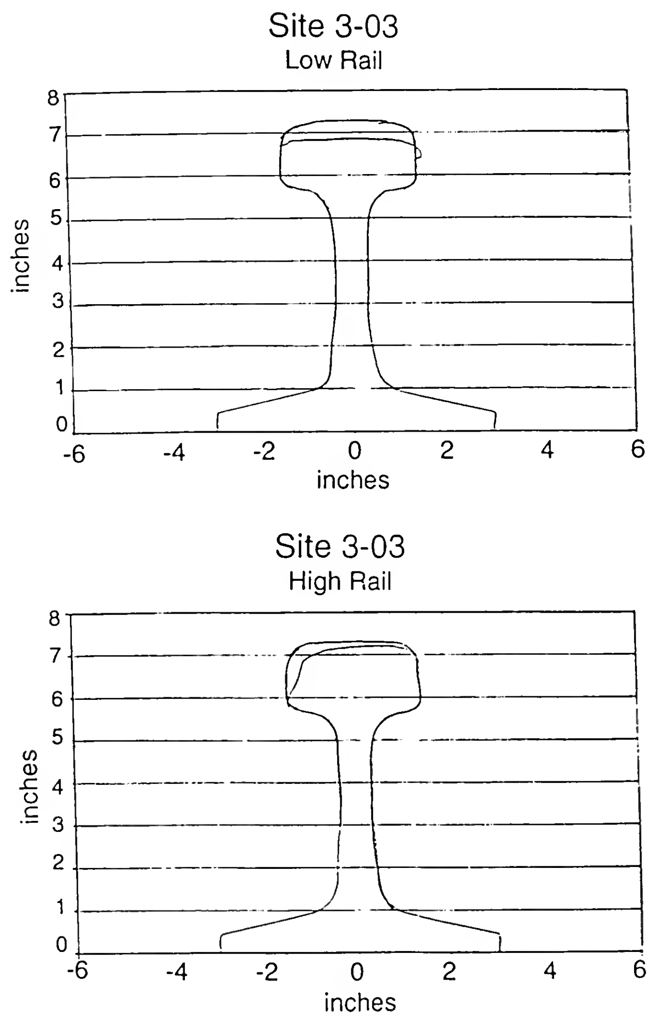

Exhibit 26. Comparison of Measured and Nominal Rail Profiles, Site 3-03.

This area was retested the same day, and the neutral temperature had increased to 91 degrees. This example illustrates the effectiveness of the TLV rail uplift system to detect "weakened" rail neutral temperature conditions. Exhibit 27 shows a photo of this location.

Several site locations were retested due to questionable neutral temperatures measured with this method. The second test was timed so that it would be done at a different time of the day, hence the rail temperature would be different. This was done in order to test the repeatability of the procedure. Overall, the uplift tests showed good repeatability, with the two sets of results being within a few degrees.

Exhibit 27. Example of Rail Cut.

7.0 CONCLUSIONS

The results of the TLV rail uplift/rail longitudinal force measurement system development, feasibility evaluation pilot tests, and subsequent revenue service field trial studies have resulted in the following conclusions:

(1) TLV-rail uplift measurement system capability and functionality proved successful through verification tests at the Transportation Test Center.

(2) Established concept implementation and system performance in providing very good accuracy and sensitivity limits. Based on pilot test data from both tangent and 5 degree curved tracks, excellent results were obtained in terms of linearity and slope, where 100% of the tangent data and 90% of the curve data fell within the 12.5 kip accuracy limit.

(3) Pilot test results enabled the development of the TLV car calibration data for measurement standardization, the development of a *"rail longitudinal force predictor"* algorithm for TLV revenue service applications, and established TLV system's adequacy for revenue service deployment for rail neutral temperature mapping studies.

(4) Overall, the rail uplift tests were extremely successful. The TLV provides a non-destructive method of determining neutral temperature of CWR, and can hence be used to inspect the track for preventative buckling maintenance. No rail needed to be cut to conduct the test, and the time required to test each location once the TLV was spotted was generally under 30 minutes.

(5) The results generally confirmed what was expected by the railroad. Any rail movement that occurred on tangent track as a result of high compressive forces correlated well with a determined neutral temperature that was below the rail temperature at the time of testing. Likewise, any movement on curves, either inward or outward, corresponded to high and low neutral temperatures, respectively.

(6) Test results further confirmed the variable nature of CWR's neutral temperature distribution, hence the need for accurate rail force measurement technique. Neutral temperatures ranged from a low of 59° F to a high of 117° F, and varied also for the two rails at the same locations.

8.0 REFERENCES

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[2] Samavedam, G., and Kish, A., *"Continuous Welded Rail Track Buckling Safety Assurance Through Field Measurement of Track Resistance and Rail Force"* TRB Research Review 1289, Proceedings of Conference on Lateral Track Stability -1991, pp 39-52.

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[4] Thompson D., et al., *"Field Testing of High Degree Revenue Service Track for Buckling Safety Assessment"* DOT/FRA Draft Report, To be published.

[5] Kalay S.F., et al., *"Track Loading Vehicle: Part 1 - System Design and Construction,"* Rail Transportation 1989, ASME Winter Annual Meeting, San Francisco, California, December, 1989.

[6] Kalay S.F., O'Donnell, W.P., *"Demonstration Testing of the Track Loading Vehicle,"* Association of American Railroads Research Report No. R-782, Chicago, Illinois.

MEMOIR

J.W. Davidson

On February 21, 1993, James W. Davidson passed away at his home in Overland Park, KS. Mr. Davidson was a former Member of AREA Committees 15 and 34, and was also a Member of the American Railway B&B Association. He had served on the Board of Directors for the latter organization on two separate occasions. He was an employee of the Burlington Northern Railroad, and the predecessor Chicago, Burlington and Quincy from 1945 until his retirement on December 31 1987 as Assistant Chief Engineer Structures.

Mr. Davidson was well known in the railroad industry as a knowledgeable and participating member both at work and in the professional organizations to which he belonged. His presence and his contributions to the industry will be sadly missed by all who knew him.

Kenneth E. Bruestle
Director Bridge Engineering
Burlington Northern Railroad

MEMOIR

William H. Pahl, Jr.

William H. Pahl, Jr., age 64, retired Assistant Vice President and Assistant Chief Bridge Engineer, Greiner, Inc., died on March 16, 1991. He is survived by his wife Marion, three children, William Wade, Richard, Lynn Chilton and three grandchildren.

Born in Lincoln, Nebraska, he served as Lieutenant (j.g.) in the United States Navy during World War II (1943-1946) in the Navy's V-12 program and served on Destroyer Escorts YMS 362 and YMS 468, performing mine sweeping operations in the Philippines, Iwo Jima, Okinawa and Tokyo Bay. He was honorably discharged in 1946 and continued to serve in the Naval Reserves until 1952.

William Pahl, Jr. studied engineering at Union College (Schenectady, New York) and received his B.S. in Civil Engineering in 1949, and was a member of the Alpha Phi of Chi Psi. He served as a junior engineer with the Pennsylvania Railroad and worked on the Connemaugh Monongehela Railroad, Office of the Chief Engineer, Pittsburgh Division.

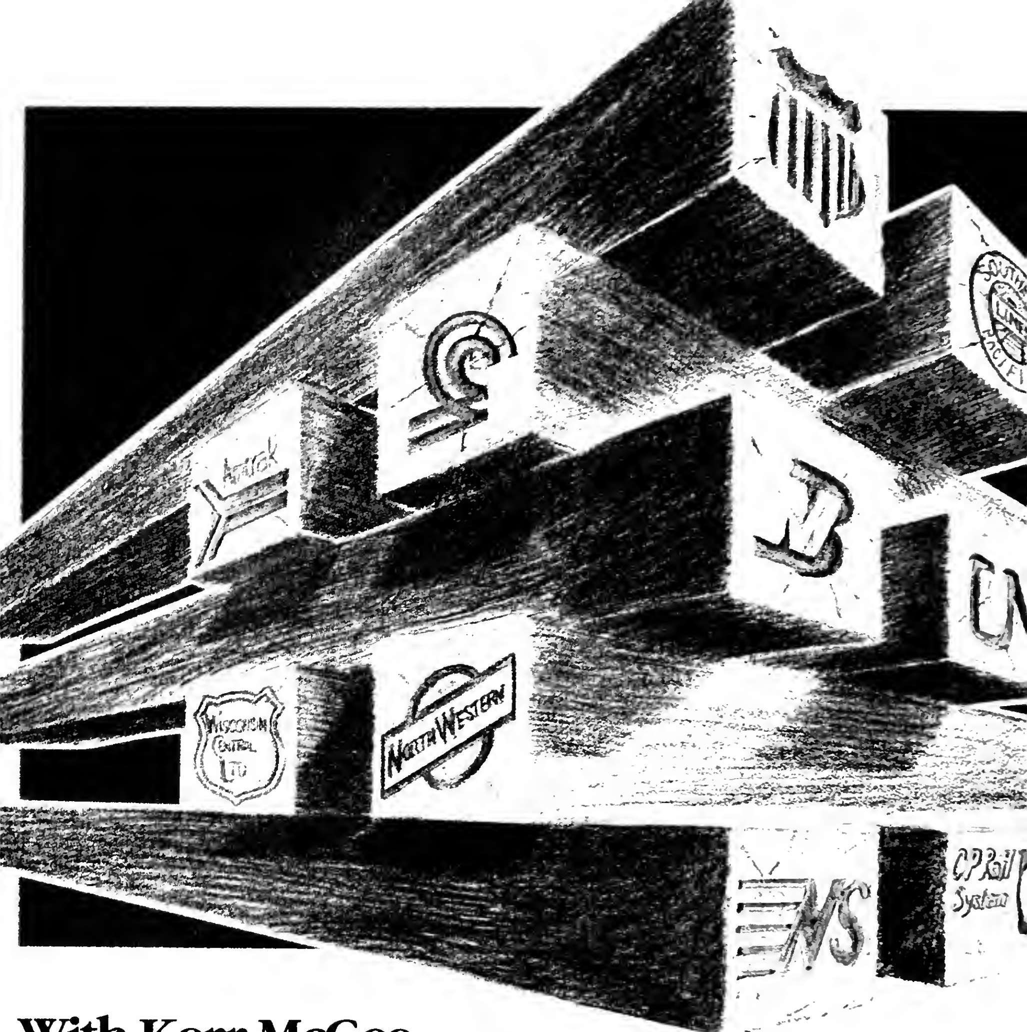
A registered P.E. in Ohio, Pennsylvania, and Maryland, Mr. Pahl joined Greiner, Inc. in 1962 and worked in its Timonium headquarters. He was specifically involved in railroad facilities projects–railroad bridges, bridges over railroads, and modifications to railroad yards and trackage. In addition, he designed and supervised study and design contracts for highway bridges and port facilities in nine states.

He became a member of the American Railroad Engineering Association in 1955 and a Life Member in 1990. Mr. Pahl was a long-time member of AREA Committee 15. He was highly regarded in the engineering profession for his technical capabilities and integrity and will be well remembered by those of us who worked with him on Committee 15.

A. E. Schmidt

NOTES

NOTES



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Front Cover: Rear helpers push grain train up eastern approach to Mullan Pass west of Helena, Montana on Montana Rail Link.

Rear Cover: Autumn 1993 scene on Montana Rail Link at west end of Austin siding, west of Helena, Montana.

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Photo 1: Large curved steel viaduct on upper portion of eastern approach to Mullan Pass on Montana Rail Link.

MULLAN PASS ON MONTANA RAIL LINK

Montana Rail Link (MRL) is a regional railroad which provides an important connection for traffic between large areas of the Midwest and Rocky Mountain regions and the Pacific Coast on its 654-mile route between Sand Point, Idaho and Billings, Montana. The most formidable obstacle on this route is Mullan Pass, which is located on the MRL main line west of Helena, Montana, capital of that state.

Photo 2: Train ascending Mullan pass about one-half mile beyond Austin siding.

Helpers are stationed at the Helena yard to assist the many grain, coal and general merchandise trains over the 2.2% grade and around the numerous curves, many as sharp as 10 degrees. The heaviest trains require both mid-train and rear helpers, requiring 13 locomotives to get over this grade. Annual tonnage on this line is 32 MGT.

The westbound grade starts just a few miles west of Helena and ascends some wide grassland valleys to the siding called Austin. Here the slope up the valley becomes too steep for economical railroad operation, and to keep the maximum grade of 2.2% the line negotiates two large horseshoe curves forming a gigantic "S" on the rising terrain (see photos above and at right). This is followed by the line running along the north slope of the valley. This stretch of track includes two large steel viaducts, 580 and 494 feet long, on 10 degree curves over side valleys (see photo on previous page).

Finally, about six miles from Austin siding, the 3,849-foot Mullan tunnel is reached at an elevation of 5,548 feet before the line descends the less demanding 1.4% grades of the west slope of the pass, which is shared by U.S. highway 12. The line was constructed in 1883 as part of the original mainline of the Northern Pacific railway. Work started on the Mullan tunnel in 1881. Expectations that the tunnel would be quickly completed vanished when soft, treacherous rock was found at the west portal. It was necessary to line almost the entire length of the tunnel with timber and to temporarily by-pass it with a line over the top with 18 degree curves and 4% grades in order to handle the first through trains in September 1883. The line over Mullan pass was eclipsed in renown when the Northern Pacific's alternate line via Butte was constructed over Homestake pass, but that line is now out of use because of its close-clearance tunnels which cannot accommodate double-stacks and because of additional maintenance problems.

In 1993, the Mullan tunnel was renovated by Montana Rail Link forces. Work included replacement of ballast, installation of a new drainage system, insulation and heating tape to prevent ice formation, and new ties with Pandrol fastenings.

Montana Rail Link's 160-person engineering department is headed by its Chief Engineer, AREA member Rich Keller, who provided much of the information for this article.

Photos 3, 4 & 5: The above two photos spread out across both pages show further progress of the train shown in photo two on the previous page as it ascends Mullan Pass. In the photo directly above, the rear helpers (with rear facing headlight) can be seen in the center of the photo, the mid-train helpers to the right, and the road engines on the front of the train at left. The top photo shows the train earlier in the ascent while the photo below shows all three levels of track on the giant "S" curve.

OFFICE OF THE CAMP WARDENS
CAMP L, TORONTO

THE RITUAL OF THE CALLING OF AN ENGINEER

The Ritual of the Calling of an Engineer has a history dating back to 1922, when seven past-presidents of the Engineering Institute of Canada attended a meeting in Montreal with other engineers. One of the speakers was civil engineer Professor Haultain of the University of Toronto. He felt that an organization was needed to bind all members of the engineering profession in Canada more closely together. He also felt that an obligation or statement of ethics to which a young graduate in engineering could subscribe should be developed. The seven past-presidents of the Engineering Institute were very receptive to this idea.

Haultain wrote to Rudyard Kipling, who had made reference to the work of engineers in some of his poems and stories. He asked Kipling for his assistance in developing a suitably dignified obligation and ceremony for its undertaking. Kipling was very enthusiastic in his response and shortly produced both an obligation and a ceremony format entitled "The Ritual of the Calling of an Engineer".

The object of the Ritual can be stated as follows: The Ritual of the Calling of an Engineer has been instituted with the simple end of directing the newly qualified engineer toward a consciousness of the profession and its social significance, and indicating to the more experienced engineer the responsibilities in receiving, welcoming, and supporting the newer engineers when they are ready to enter the profession.

The Ritual is administered by a body called the Corporation of the Seven Wardens. The seven past-presidents of the Engineering Institute in 1922 were the original of seven wardens. The Corporation is responsible for administering and maintaining the Ritual, and in order to do so creates Camps in various locations in Canada. The Ritual is not connected with any university or any engineering organization; the Corporation is an entirely independent body. The Ritual has been copyrighted in Canada and in the United States.

The Iron Ring has been registered and may be worn on the little finger of the working hand by any engineer who has been obligated at an authorized ceremony of the Ritual of the Calling of an

Engineer. The ring symbolizes the pride which engineers have in their profession, while simultaneously reminding them of their humility. The ring serves as a reminder to the engineer and others of the engineer's obligation to live by a high standard of professional conduct. It is not a symbol of qualification as an engineer - this is determined by the provincial and territorial licensing bodies.

UNION PACIFIC RAILROAD HASTINGS BYPASS PROJECT

Tom Ogee*

I would like to describe the planning and design of the Union Pacific's Hastings Bypass Project, but first let me describe a little bit about our operation through this area and the history of the project. The Union Pacific's major east-west corridor passes through Nebraska and the historic route of the Union Pacific started in Council Bluffs, Iowa crossing the Missouri River into Omaha and then on to our major classification yard located at North Platte. But also at Gibbon, Nebraska our Marysville Subdivision departs from this line and continues southeasterly through Hastings and into Kansas and on to Kansas City. This track was constructed in 1914 and for many years functioned as a secondary line. But in the last 20 years this route has become more and more important as a major traffic corridor for us. Last year 106 million gross tons were transported over this predominately single track railroad. This year we are averaging 50 trains per day. We have been steadily progressing capacity improvement projects on the Marysville sub line to handle our tremendous growth in traffic. Since 1989, just in this area, we have constructed over 17 miles of 2nd main track at a cost of $19 million.

Public concern over train congestion and road crossing blockage in Hastings is not new. Our records show that in the late 1930s the city studies the possibility of constructing a grade separation. but neighborhood conflicts stopped the project. In the early 1970s the Union Pacific proposed a second main line on the existing track alignment through Hastings. This proposal was eventually shelved because of traffic congestion on the numerous road crossings through town. In the late 1970s, railroad relocation scenarios were discussed with the intent of qualifying as a federal demonstration project under the 1973 Federal Aid Highway Act, but enough support , both financial and political was never obtained. During the 1980s, all crossings through town were improved with traditional signalization methods which substantially improved safety, but did not address the congestion issues, both highway and rail. then in 1990, a 7th Street grade separation bond issue was presented to Hastings residents and passed. By late 1990 final engineering drawings were being prepared for 7th street and UPRR officials assured the City that a reroute around town would not happen.

However, during this same period, the Union Pacific was experiencing significant growth in our business, primarily transporting coal from the Powder River Basin to eastern utility customers. In response to this traffic growth, we again reviewed capacity constraints along this corridor. On January 2, 1991 the Union Pacific announced a $250 million plan to expand the primary coal route from the Powder River basin coal fields in Wyoming. Then on February 2, 1991 just five months after announcing we weren't planning on relocating, we announced that plans were underway to relocate our track from downtown and construct a double main line bypass. Showing once again what a dynamic and ever changing business railroading has become.

Our track through Hastings is shown in red on this slide. The Burlington Northern also goes through Hastings and they are shown in green. We interchange traffic with the BN in Hastings as we both have customers located in town. So key to any of our plans was to find a way to maintain service and find a new way to interchange with the BN. As you can see, we cross the BN at grade right in town which is a major point of congestion and delay. As stated earlier, we have 50 trains a day and the BN was 10-15 trains per day all trying to get through this crossing frog. Our maximum speed through town is 30 MPH and if we stop at the hold signal for the xing frog., we block the

*Manager—Facilities Planning, Union Pacific Railroad

streets in town. To hold this blockage to a minimum while waiting to get across the diamond, we hold trains back, but crossing blockage is still a problem.

The route that proved to be most feasible for us was the one show in purple on this slide. To avoid an at grade xing of the BN's Aurora Branch, we are providing anew connection for them which is shown in orange on the right side of the slide. We are also constructing a couple of new tracks at the Hastings Industrial Park adjacent to the existing track shown on the far right to accommodate the UP-BN Interchange.

Right of Way Acquisition

Once the decision was made to pursue the line change, the first order of business was to tie down the alignment sot hat property acquisition could start. Obviously, substantial amounts of new right of way were required to build the project. Preliminary engineering indicated a 200 foot. wide strip of land was needed through the entire 51/2 mile project. Final design remained at 200 ft. with the exception of the one big cut where 300 ft. was required.

One of the first parcels to be acquired was property owned by the Crosier Monastery at the northern end of the relocation. Friar Jim Moeglein and the rest of the Crosier Fathers were very helpful but yet concerned about the effects the project would have on the monastery grounds. We had a sound and vibration study done for them, and then to help them better understand how we would address their concerns about noise, we had an artist's rendering done showing an earth berm with trees on top that we agreed t place between the railroad and their monastery grounds. This right of way acquisition was essential to getting the project underway and set the tone for all subsequent negotiations. This is what it looks like with the berm in the center, the monastery grounds on the left and the rail grade on the right. Fortunately there was a tree farm on land that we needed so we had a ready source of fir trees.

The total railroad operating right of way required for the project is approximately 140 acres. However through the negotiations 56 parcels were acquired amounting to nearly 570 acres or more than 3 times what we actually needed.

Relocation of Utilities

Major private utility relocations were necessary throughout the 5.45 mile project. A total of five private utility companies were involved with approximately 60 individual utility crossings to be addressed. This expenditure alone will total more than $2.25 million by the completion of the project.

Some of the larger utility relocations included two 115 KV electrical transmission lines owned by Nebraska Public Power District and Hastings Utilities shown on the left on this slide. Tow 34.5 KV electrical liens owned by Southern Nebraska Rural Power District. A 36 inch sanitary sewer (shown in the foreground), a 10 inch water line owned by Hastings Utilities and a fiber optics line owned by Lincoln Telephone.

The negotiations with the individual utility companies were more time consuming than originally anticipated. The sometime strained relationships between utility companies and railroads caused delays in the utilities design and engineering time frames. The willingness of utilities to respond to our accelerated construction schedule was also a concern. But in the end everyone cooperated and the utility work eventually proceeded smoothly.

Environmental Issues/Wetlands

Environmental issues and permitting are always a concern and usually an unknown in larger projects. This project was no exception, although it did not become a problem. The alignment passes through six jurisdictional wetland areas, totaling 4 acres. Additional projects we are doing in the area added 21/2 acres for a total of 61/2 acres of wetland that we needed to mitigate.

The sensitivity of the wetlands was further heightened by the presence of a Rainwater Basin area. The Rainwater Basin was jointly designated by the U.S. Army Corps of Engineers, U.S. Fish & Wildlife Service and the Nebraska Games & Parks Commission. This special status was created because of the central flyway for migratory birds such as sand hill cranes. The Rainwater Basin is especially important during the spring migration as the basin is full of shallow wetlands that promotes a high protein food source to strengthen waterfowl for their continued flight north.

Recognizing these factors we decided to propose creation of 2 acres of new wetlands for each acre of existing wetland filled as a good faith effort on our part to restore wetland in this environmentally sensitive region.

Subsequent field investigations uncovered an existing 5.1 acre wetland near the project site that had been drained for agricultural purposes that could be reestablished to satisfy our proposed mitigation. This site was located on private property adjacent to the project and we purchased the property and prepared the final mitigation plan that was accepted by the government agencies. The final mitigation site totaled 21.7 acres comprised of 12.3 acres of seasonal wetland, 5.7 acres of temporary wetland, and 3.7 acres of buffer area surrounding the site. We are required to monitor the effectiveness of the mitigation site for a period of 5 years to ensure that the wetlands function properly, but with the se weather we've been having it should be easy to get them started. We are considering an offer made by an environmental group, Pheasants Forever, who would like to manage the wetlands for us.

Civil Engineering Design

The final track alignment was selected from a number of different alternate routes that were considered. We reduced these down to 4 alternates that we evaluated in detail before selecting the alignment that is now under construction. Starting on the north end or the left side of the slide, the grade varies from 0.5% to level for approximately 2 miles which produces adequate embankment for approaches to the Elm Street grade separation and backfill for the large twin concrete box structure at the Lake Heartwell Tributary. It then goes up on a 0.83% grade to meet the BN and Highway 6 overcrossings. South of the Highway 6 grace separation the alignment returns to natural ground on a 0.78% grade and then continues south on a .3% grade and ties into the existing track alignment.

The maximum horizontal curve through the line change is 1 degree 29 minutes with 60 MPH spirals. The roadbed is 60 ft wide to accommodate the double track at 20 ft centers with 15 ft shoulders and a 10 ft maintenance road. All of the rail on both tracks will be new 133# head hardened continuous welded rail on concrete ties. On each end were double main line track returns to single main, we are installing No. 30 turnouts with the tangential geometry and movable point frogs on concrete ties which will allow 60 MPH operation through the turnout. Each turnout requires 3 switch machines. Two on the points which are 1151/2 feet long and one on the movable frog. The total length of the turnout is 375 feet. A universal crossover consisting of 4, what we call premium, No. 20 turnouts with movable point frogs and tangential geometry on concrete switch ties will be installed on the south end of the project near KICS Road.

The alignment and grades have been designed for 60 MPH operation compared to the 30 MPH on our existing line.

Project Coordination

Our goal from the outset was to build this project just as quickly as possible. This would not have been possible if it were not for the total cooperation and coordination of everyone involved. This project included nearly every department on the Union Pacific. It has been a real Team effort. Contracts & Real Estate, Law, Government Affairs, Environmental Management, Public Relations, Planning & Analysis and all areas of the operating department have all been closely involved with the project.

On the public side, negotiations for public funding of the highway grade separations began at the very beginning. The City of Hastings, Adams County and the State Department of Roads all cooperated and it was soon agreed that the monies that would have been used for improvement son the existing alignment would be used for the three new structures. One belongs to the city, Elm St., and the other two are the state's, Hwy 6 and Hwy 281. This required very close coordination as the Federal funding we are using required us to adhere to all of the requirements of the FHWA on property acquisition and relocations just as if this were an Interstate Highway. We learned quite a bit about documentation and red tape. This also led to a unique arrangement where our consultant designed and prepared the plans and specifications for the three highway bridges and the State solicited bids, awarded the contracts and they are administering the contracts. We also reached an agreement with the BN where we would design and have constructed the new UP railroad bridge over them.

Construction

By this point in the project everything was moving smoothly and we were on a fast track. The first contract for clearing and demolition work was let in the summer of 92. By the time winter weather forced us to shutdown in December, all of the major contracts has been awarded and more than 400,00 cubic yards of embankment had been placed. The severe winter and a late spring kept us from staring significant work until May 93. Then just when we were getting going good, the Midwest's summer of rain and winds started. A near tornado force wind one night caused this damage to one of the cranes as well as other damage on the project. And just like everywhere else in the region, the rain devastated the project. In June and July more than 15 inches of rain fell on the project. The rains have continued through August although not nearly as heavy, and we are just now getting back to full production on the grading. Our current schedule calls for the grading and bridges to be finished by the end of October. We plan on laying the track using two construction machines placing concrete ties and threading the rail up on them starting in late October. The track and signals are scheduled to be in service by the end of the year with final completion and cleanup to be finished in the spring.

When we're finished the project will have cost $30 million. We will have added 51/2 miles of double track CTC railroad, eliminating the UP-BN at grade interlocker and eliminating 11 at grade railroad crossings And we will have improved the speed from 30 to 60 MPH.

We shudder every time we hear a long range weather forecast calling for an early and severe winter, but we'll keep pushing as hard as we can. When I see you around the coffee pot at the Spring Conference I'll let you know how mother nature treated us this fall and winter.

Experience with Railroad Bridges and Projections for the Future

by K. J. Norton*

REMSA's extremely proud of the display they have assembled in Denver this week. With representatives attending from the entire world, it certainly can be considered a global event. I encourage you to get over to the show, ask questions, snoop around, kick some tires and challenge us who make up the supply business to help you solve some of your problems. I encourage you to also point out ideas for improvement. We need you and you need us, and together we can make certain that industry prospers. We are not, after all, an immense industry. Railroads and their supply partners combined represent an industry totalling $34 billion. The pizza industry is a $29 billion industry. So we must work together in unison.

"Experience With Railroad Bridges and Projections for the Future."

That's a great title! Many of you in this room would be much more qualified than I to make this presentation. Perhaps the only advantage I have is a wider perspective, encompassing the entire industry, as opposed to one specific railroad. Perhaps a wider perspective...but certainly less detailed.

My experience seems to tell me that there may be storm clouds gathering on the horizon with respect to the industry's bridges. Indicative of this, I had a conversation lately with an individual who shared with me the following information regarding his bridge population. His railroad currently has: 22 miles of concrete bridges exceeding 65 years, 30 miles of steel bridges exceeding 85 years, 37 miles of timber bridges exceeding 65 years. This situation mirrors, I believe, the entire industry. We are an industry with an aging bridge population. This situation, however, needs to be kept in perspective, since to date there has not been a major bridge failure. The perspective is, yes, there is an aging bridge population, but up until now this has not been a problem.

My experience tells me that the industry's bridges have served well, but I also see storm clouds on the horizon. My experience tells me that the designers of these bridges built into their plans excess capacity and some redundancy and unknowingly consideration for the future. My experience also tells me that this was not planned...it just happened! I might add, it's a darn good thing that it did!

There is little, if any, of the same redundancy built into structures designed and constructed today. This situation is even more true when it comes to highway bridges. Most highway bridges today lack the capacity to even compensate for the deterioration process due to aging, and yes there have been bridge failures on the highway side.

I am told that timber bridges are supposed to last for 50 years, yet the typical timber bridge in the railway industry today substantially exceeds this age. The average age of timber bridges on some railroads today even exceeds 70 years. Not all of these bridges have been subjected to in-place retreatment to extend their life...perhaps another storm cloud on the horizon. Many concrete bridges exceed 75 years of age and no one can tell me just how long a concrete bridge is supposed to last. I hear lots of people say 65 or 75 years. In many cases, it is not the entire portion of concrete structures which succumb to the aging process. It is specific locations, areas of high stress where points of failures begin to occur, such as bearing areas.

Steel bridges are in use today that are equally as old. How long are they supposed to last? I wonder about that! I believe it was AAR's Al Reinschmidt who said, and I quote, "Loading cycles accumulate on the fatigue clock and at some point you get to midnight." Interesting...I wonder where we are now?

*Vice President, Railroad Engineers Maintenance Suppliers Assoc.

The service life that the railway industry has been able to obtain from their bridges is nothing short of amazing.

I ask myself "why?"

I wonder if the answer might be:

- Quality engineers that were the backbone of this industry.
- Quality oriented work crews that were responsible for construction and maintenance.
- Quality products used in the construction process.
- Frequent inspections by individuals who had that as their only responsibility.

No one can deny that the situation has changed, and I wonder what the effects of this will be. I wonder too how long it will take for the effects to surface. Another storm cloud? Certainly, the industry still has quality engineers, but what about the drastic reduction in their numbers? You know better than I that there has been substantial downsizing of engineering departments, to the point where a large portion of engineering is now done "outside" by consultants. This, in and of itself, may not be bad; but I simply wonder what affect it will have on the ability of the industry to continue maximizing the service life of their structures.

What about the quality oriented work crews that existed in the past? What about the reduction in their number? A large portion of bridge repair work is now being handled by outside contractors. Much of the work involved is awarded on low bid. I wonder if the industry is getting the same level of quality work, given this situation, as it did in years past and which was a major factor in allowing the industry to maximize the service life of their structures.

I wonder about the long range service life of many of the products going into structures today that are also purchased on low bid.

I wonder particularly about bridge inspections and even more specifically, bridge inspectors. Years ago there were individuals who were assigned bridge inspection as a sole responsibility. These individuals got to know the bridges they were responsible for intimately. They could see trends and they could detect changes. Throughout my career, I've had the privilege to spend considerable time with a large number of bridge inspectors. Like all of us, they came in various shapes and forms, some good, some bad, some not so good. But all were intimately familiar with the bridges they were responsible for. Frankly, I learned a lot from these individuals. I remember an individual on the former Frisco who each morning would take off in his motorcar and inspect bridges throughout the day...day in...day out...good weather, bad weather and may have returned to the same structure on a monthly basis. That's what I mean by getting to know the bridges intimately and being able to detect even minute changes. This inspection system allowed preventive maintenance to be accomplished in a timely fashion and undoubtedly maximized the service life of structures involved. Many of these same people are not around today, their numbers have been decreased substantially, and I wonder what the effect of this will be.

I wonder about the 125 ton car. Should the fact that our bridges accepted the 100 ton car allow us to automatically assume that this situation will be the same for the 125 ton car?

I wonder about economic and financial philosophies that exist today on some railroads where there is an emphasis on short term financial gains and a corresponding de-emphasis on long term gains to the point where preventive maintenance does not always fit into the financial formula. The industry's bridges are in the shape they are in today because, for the most part, the industry was willing and interested to spend a dollar today to save two tomorrow. I suppose it's possible to adapt a financial outlook that states that I will not spend anything today in order to maximize current return, but I wonder if the money will be available tomorrow to complete this formula? I wonder too where the individuals will be tomorrow who formulated this financial plan?

I wonder about railroads with high timber bridge populations. With so many bridges all the same age, there will be a large number of structures requiring rebuilding in a short time frame. Will there be funds available for this? It simply is not realistic to expect timber bridges that have not been in-place retreated to continue doing their job when they have reached an age of 80 or more years.

No one can deny the fact that the industry has an aging bridge population. This is true for timber, concrete and steel structures. This fact alone dictates that this asset needs to be micro managed if we are to continue to expect the kinds of service and safety these structures have provided the industry. There must be sufficient funds for maintenance and replacement, on an annual basis.

I wonder about the bridges on the nation's 475 short line and regional railroads. A high proportion of the industry's lighter capacity structures are located on these railroads just by virtue of the way these organizations were formed. They are now short line and regional railroads. They were light density branch lines on Class I railroads, and as such have many of the older lower capacity bridges. Many of these railroads are highly leveraged and may not be able to spend an appropriate amount of money attacking the problem of an aging bridge population and heavier loads.

I wonder about a possible "ripple-effect" that might impact Class I railroads if a negative bridge situation were to occur on a short line or regional railroad. Will a problem here generate federal involvement in one form or another? Federal involvement that will spill over and affect the larger Class I railroads. In my opinion, this is possible and if it is, the Class I railroads have a substantial vested interest in bridge condition and bridge inspection on short line and regional railroads, and I wonder how we address that problem.

I wonder too if the current level of federal involvement, mostly in the area of bridge inspection, will remain static. I don't know of many cases where this has been the situation. We all know the propensity of the government to increase involvement whether or not it is needed. Where is this initiative headed? What's the agenda?

We know that the Federal Railway Administration has already established bridge safety standards. My firm was pleased to have been consulted prior to finalizing these rules and regulations, and we considered it critical to have had a voice in arriving at something which is acceptable and workable in this area.

If you have detected that I am wondering about a few things, and about a few situations, you are correct. I reiterate that the industry's bridges have served well. Conditions have a habit of changing, however. Nothing seems to stay the same. There have been more changes in industry in the past 5 years than in the previous 20. In 1988 alone, there were 3500 merges and acquisitions. Since 1980, 40% of the Fortune 500 Companies have disappeared. In 1991, 187 banks went "belly-up". Ten years ago, who would have thought that sneakers would cost $100 and even have lights in them. I suspect even the most astute marketing analyst for Keds or some other sneaker manufacturer would have been laughed out the industry 10 years ago if he put forth these ideas. I simply serve these up as examples of drastic changes that can occur.

Yes, the industry's bridges have served well, but there are some storm clouds on the horizon which, in my opinion, need to be addressed.

There is another interesting and yet subtle trend occurring in the industry today affecting bridge maintenance and reconstruction. Specifically, that work which is accomplished by contractors. For years railroads developed their own repair specifications. A few years ago when down sizing (we now call it re-engineering) first hit engineering departments, we saw repair and rebuilding specifications go outside and generated by consultants. Now we're seeing a trend where contractors are not only asked to accomplish the required work, but also design the repairs involved. This means that we, the contractors, are having to increase the size of engineering staffs in order to accommodate this requirement. It's nothing new, it has been around for a long time, and has frequently been referred to as design/repair or design/build. Nonetheless, it is changing the way we conduct our business and means that we must now place equal emphasis on design in addition to actually getting the job done. For the railroads, it will probably mean a slight upward trend in repair costs, since pricing will obviously have to reflect a portion of our engineering costs.

So, what in my opinion, must the industry do to assure continued safe service from its bridges?

1. Continue to dedicate itself to timely, accurate inspections of bridges...all bridges.
2. Recognize that an aging bridge population is oblivious to corporate financial strategies. Maintenance and structural repair situations arise on bridges irrespective of current financial philosophy.
3. Continue to accomplish preventive maintenance on a priority basis.
4. Ask for and insist on safe, quality work from railroad forces and contractors alike.
5. Allocate the time to dig deep into the work done by contractors to make certain work is being accomplished according to specifications and at the highest possible level of work quality.
6. In comparing bids from contractors, give equal weight to project specifications, material quantities and past work history as to price.
7. Continue to "lobby" hard for maintenance and capital. Budget commensurate with the needs of your specific railroad and for maintenance programs that recognize that we are dealing with an aging bridge population.

And lastly,

8. Be willing to give advice, share ideas and assist where possible your transportation partners—the regional and short line railroads.

Ladies and Gentlemen—Thank you very much!

INVESTIGATION OF AN OPEN DECK THROUGH-TRUSS RAILWAY BRIDGE: REVENUE TRAFFIC TESTS

D. H. Tobias*
D. A. Foutch**
J. Choros***

This article describes tests of an open deck through-truss bridge under revenue traffic. The objectives of this test were threefold. The first was to develop a spectrum of wheel loads at the site. This will be added to similar data collected at other sites in order to develop a better understanding of the loads that bridges are required to carry under today's loading environment. The second objective was to measure the dynamic stresses developed in selected members under this loading environment. The third objective was to demonstrate the use of field measurements in predicting the remaining life of railway bridges.

The main span of the bridge is an open deck Warren truss, 156 feet 3 inches across Big Creek. It was built in 1919, with an original design loading of Cooper E60 with steam impact, and is located near the town of Del Rio in the southeast corner of Tennessee. It is on a single track Norfolk Southern line that runs from Knoxville, Tennessee to Ashville, North Carolina, at milepost 195.3. Of the 17 MGT the line carries annually, about 11.8 MGT crosses in the Knoxville-to-Ashville (eastbound) direction and 5.2 MGT crosses in the westbound direction. Approximately 50 percent of the traffic is unit coal traffic and the rest is mixed freight with a small portion of intermodal. On the average, there are three unit coal trains and four mixed freight trains that cross the bridge each day.

Chord members and diagonals are latticed built up sections and hangers and posts are solid built up I-sections. The floor system consists of two stringers on 6 ft.-6 in. centers which are made from built up I sections. The stringers span 26 ft.-1/2 in. between floor beams that are also built up I-sections. A photograph of the bridge is shown in Exhibit 1.1 and a photograph of a floor-beam-to-truss connection is shown in Exhibit 1.2. A schematic of the bridge is shown in Exhibit 1.3.

Exhibit 1.1

*Ph.D. candidate, University of Illinois
**Professor, University of Illinois
***Section Manager, AAR

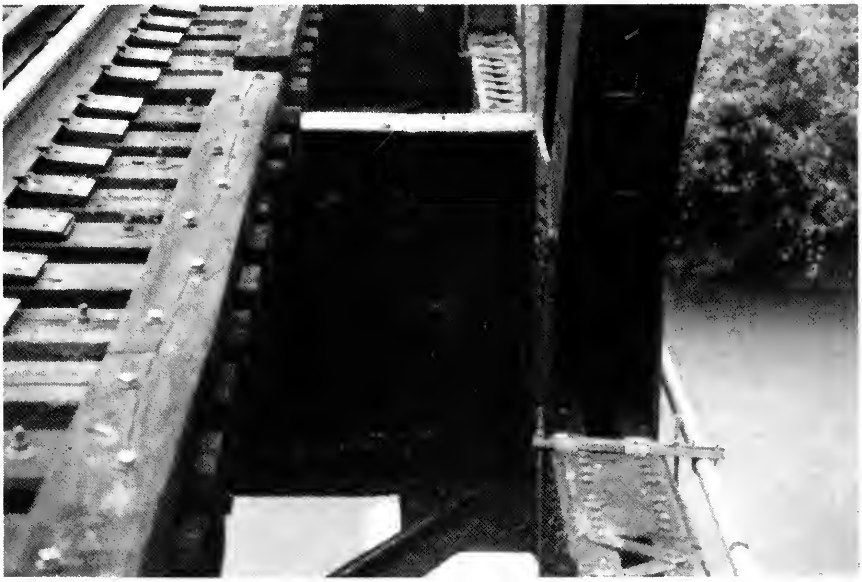


Exhibit 1.2

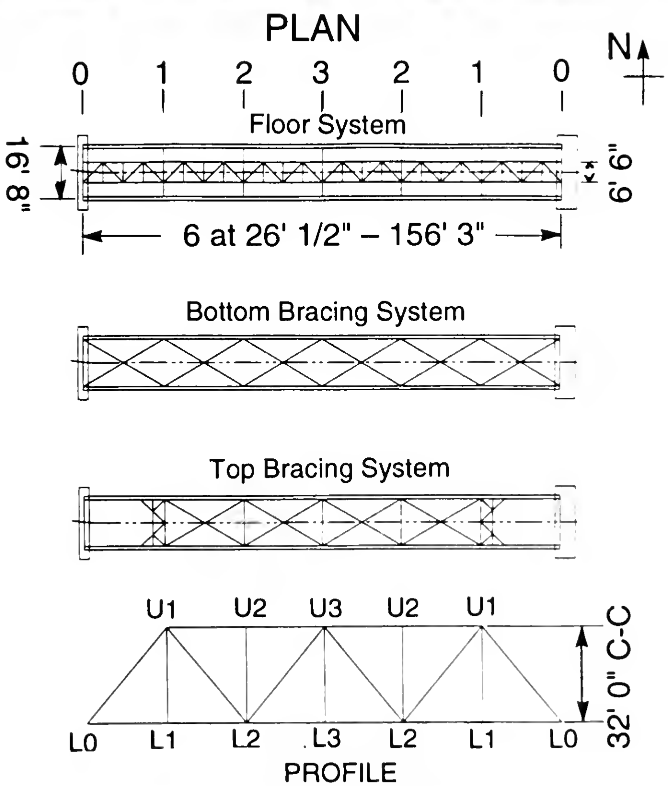


Exhibit 1.3. Plan and Elevation Views of the Bridge

The 132 pound continuous welded rail is box anchored to each tie. The ties are 10 ft. long by 10 in. square and are placed on 18 inch centers. The tie plates are 18 in. wide with one cut spike on each side. There is a 25 mph speed limit due to a 5 degree left hand curve for eastbound traffic that begins within a few feet of the west approach to the bridge.

This investigation concentrated on the western half of the bridge. During these tests, over 180 different measurements were taken. Most of the measurement points were located near truss and floor system joints. Mid-span moments were also measured for selected floor beams and stringers.

After completion of the work train tests, the measurement locations were condensed to a set of 60 locations to accommodate the data acquisition systems that were placed on the bridge. Exhibit 1.4 shows the measurements locations on the north and south trusses that were used for the revenue traffic tests. In some cases, axial strain as well as in-plane and/or out-of-plane bending strains were recorded. In others, only axial strain was measured. A significant amount of the instrumentation was placed near L1 and U1 on both trusses, because these areas have been identified as problem spots for fatigue on other trusses. Exhibit 1.5 presents measurement locations for the floor system and bracing members. Stringer measurements were concentrated between L2 and L3 where end-of-stringer bending strains were measured. Similar strain measurements were recorded for floor beams at L1 and L2.

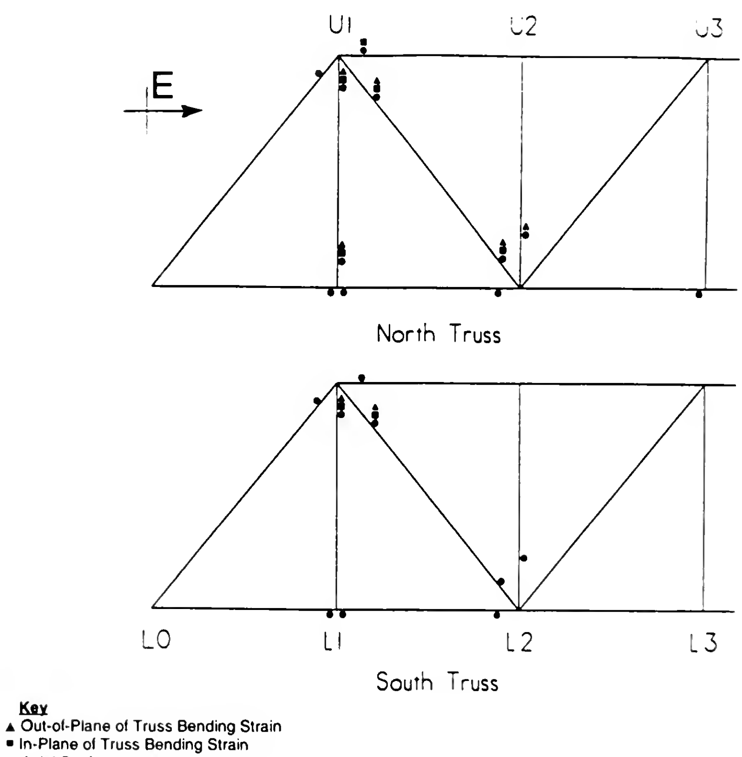


Exhibit 1.4. Location of Measurement Points for the North and South Trusses

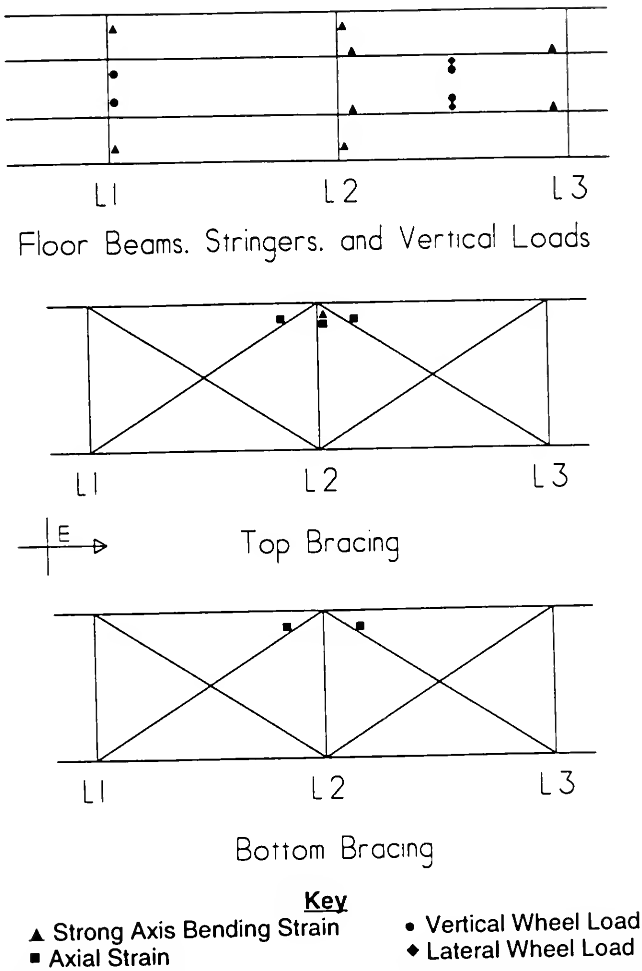
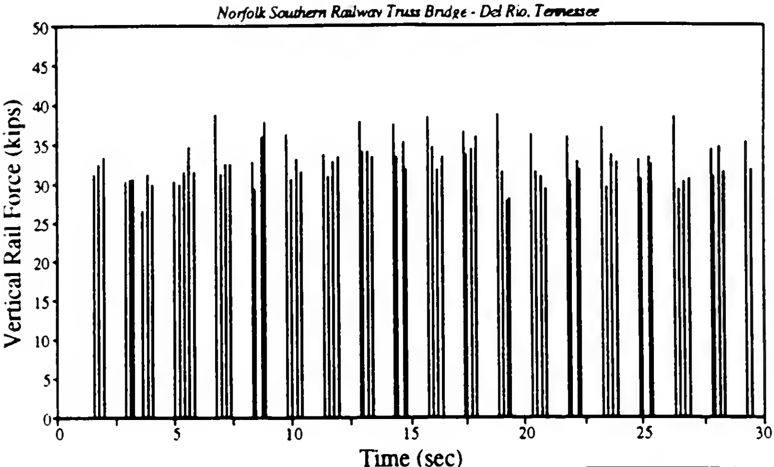


Exhibit 1.5. Location of Measurement Points for the Floor System and Bracing

Vertical wheel loads were measured on the north and south rails at L1 and at the midpoint between L2 and L3. Lateral wheel loads were measured on the north and south rails only at the L2-L3 position.

Exhibit 2.1 shows a 30-second window of the vertical wheel loads measured at L1 for a typical train passage. Although the data was collected continuously at 300 samples per second, only the peak wheel forces are shown in Exhibit 2.1. This mixed freight train crossed the bridge travelling eastward at about 26.7 mph. The train was composed of four locomotives and 101 trailing cars. The total weight was 14,298 tons and the trailing weight was 13,489 tons. The average wheel load for the trailing cars is 33.4 kips and the average axle load is 66.8 kips.

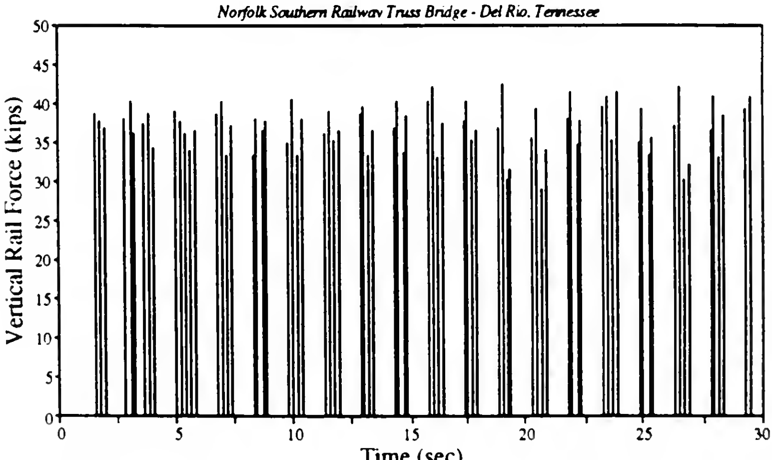
North Rail at L1



Date: Feb 10, 1991
Time: 1:07 am
Type: Mixed Freight
Ave Sp: 26.7 mph

Tot Wght: 28595 kips
Trl Wght: 26978 kips
No Locomotives: 4
No Trl Cars: 101

South Rail at L1



Date: Feb 10, 1991
Time: 1:07 am
Type: Mixed Freight
Ave Sp: 26.7 mph

Tot Wght: 28595 kips
Trl Wght: 26978 kips
No Locomotives: 4
No Trl Cars: 101

Exhibit 2.1. Time History of Vertical Rail Forces on the North and South Rails at L1 for a Typical Train Crossing

A visual comparison of the forces measured on the north and south rails reveals that, on average, the wheel loads on the south rail are a little larger than those on the north rail. The reason for this is that the train is coming off a 5 degree left hand curve as it travels eastward. This is consistent with results from previous tests [1] which revealed that members in the south truss carry loads that are sometimes 10 percent higher than similar loads in the north truss for eastbound traffic.

This train was heavily loaded, and many of the wheel loads approached or surpassed 40 kips. It is also clear from the data that there is a large variation in wheel loads for a given car. This is primarily due to the dynamic response of the car as it travels along the track but it may also be due to unevenly loaded cars and/or unequal distribution by the three-piece truck suspensions.

For each train passage the wheel load and axle load data are processed and stored in histogram form. Exhibits 2.2 and 2.3 show time histories of the axle forces measured at L1 and at L2-L3. The histograms for the north and south rail forces for the typical train passage are shown in Exhibit 2.4. The histograms of axle forces for L1 and L2-L3 for the same train passage are shown in Exhibits 2.5 and 2.6, respectively.

These histograms confirm that the south rail forces are greater than the north rail forces. Although there are some differences in load counts in the top three load ranges, the histograms of axle loads reveal that the axle loads measured at L1 and at L2-L3 are very comparable. Although the leaning of the cars due to centrifugal forces as they travel through the curve causes the south rail forces to be larger than the north rail, the effect on the axle load is minimal since the increase in one wheel force is compensated for by a decrease in the force in the other wheel.

Train passages were monitored during the months of February 1991 through June 1991, and data was processed for 81 train passages. Histograms of vertical wheel forces measured on the north and south rails at L1 for the 81 train crossings are shown in Exhibit 2.7. The data indicates that the wheel load distribution at the site is nearly bimodal. The large number of wheel loads per train crossing in the 5 to 15 kip range indicates that a large percentage of the traffic consists of empty cars. These are likely to be unit coal trains returning to pick up coal. The data also indicates that more occurrences of loads above 35 kips occur on the south rail than on the north rail. This is probably due to the effects of the curve on eastbound traffic.

Wheel load data for the L2-L3 location are shown in Exhibit 2.8. The bimodal nature of the wheel load distribution is clearly evident here as well. Here, however, the north rail seems to get a few more loads above 35 kips on the average than the north rail. This could be the result of the cars rebounding as they come off the curve in the eastward direction.

Histograms of the axle loads measured at the two locations are shown in Exhibit 2.9. As expected, there is less difference between the axle loads measured at the two locations than there is for the wheel forces. The adding of the wheel loads to get axle loads tends to eliminate the effects of the curve and reduce the fluctuating effects of car dynamics.

The best representation of the wheel load environment at the bridge site is obtained by averaging the wheel loads measured at all four locations for the 81 train crossings. The result is shown in Exhibit 2.10. The average axle loads at the two locations are shown in Exhibit 2.11.

The average train crossing the bridge has 320 axles. This could be represented by 2 six-axle locomotives and 77 trailing cars or by 4 six-axle locomotives and 74 trailing cars. Seventeen percent of the wheel loads exceed 35 kips; 3 percent exceed 40 kips; and about 3 trains in 4 has a wheel load that exceed 45 kips. For the axle loads, 14 percent exceed 70 kips, but only 0.2 percent exceed 80 kips. Only about one train of 50 has an axle load that exceeds 90 kips.

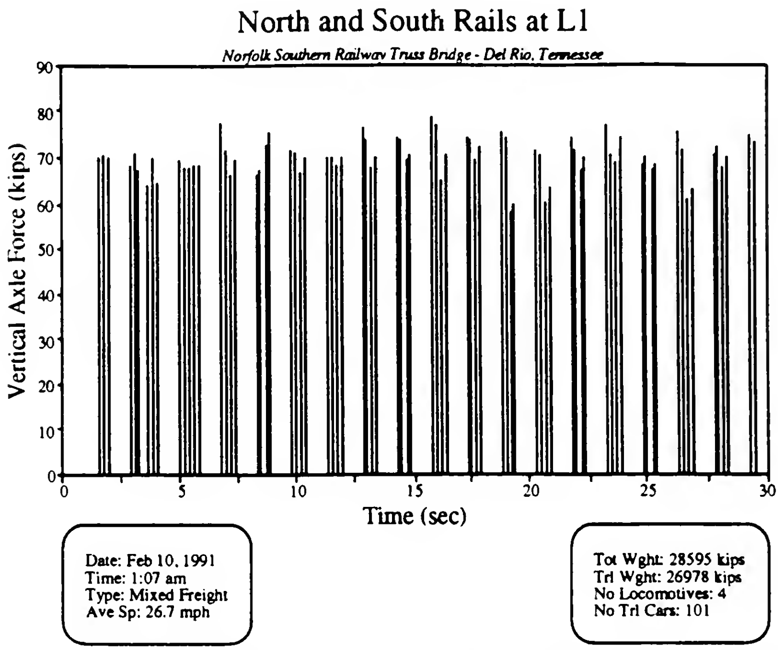


Exhibit 2.2. Time History of Axle Loads as Measured at L1 for a Typical Train Crossing

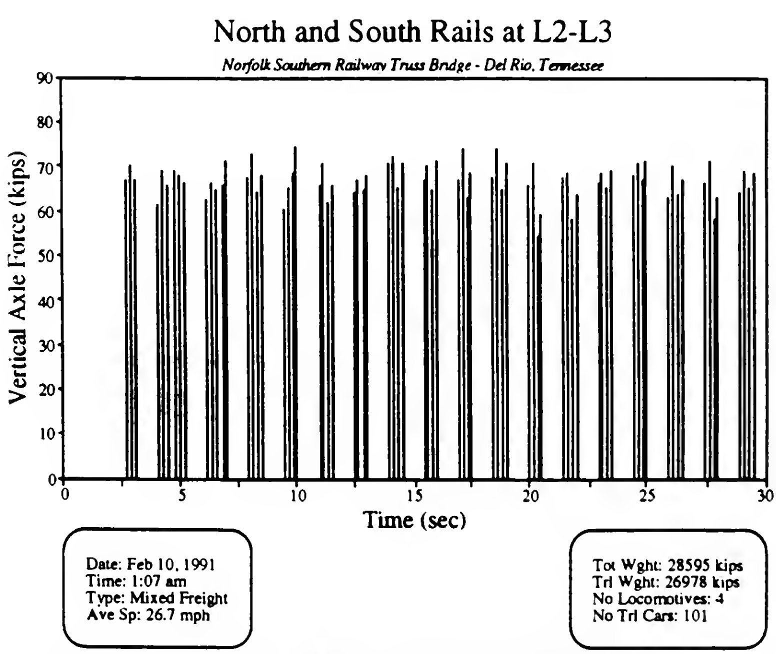
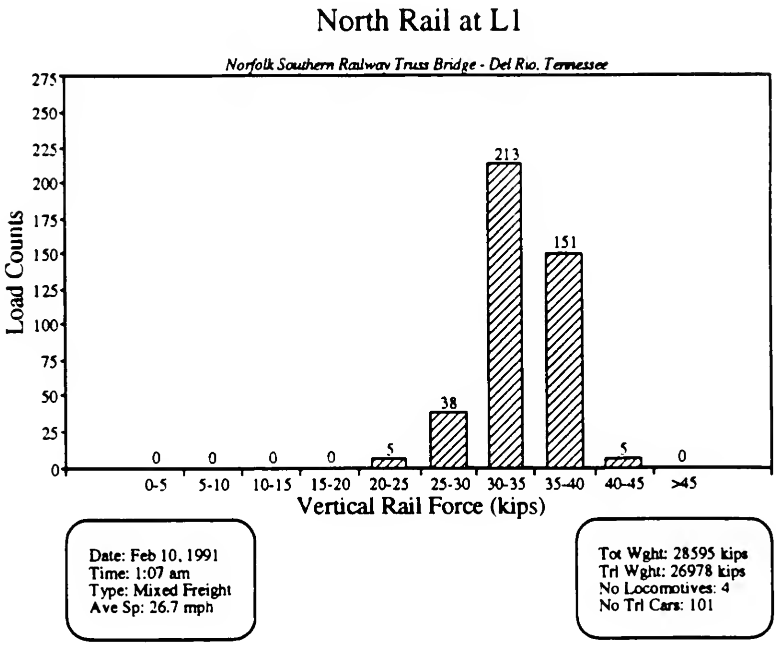


Exhibit 2.3. Time History of Axle Loads at L2-L3 for a Typical Train Passage



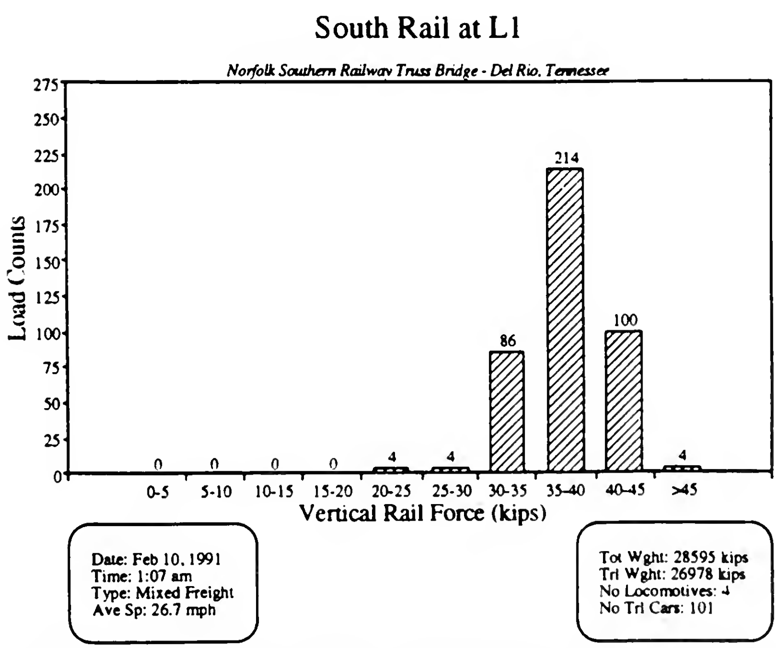


Exhibit 2.4. Histogram of Vertical Wheel Forces on the North and South Rail at L1 for a Typical Train Crossing

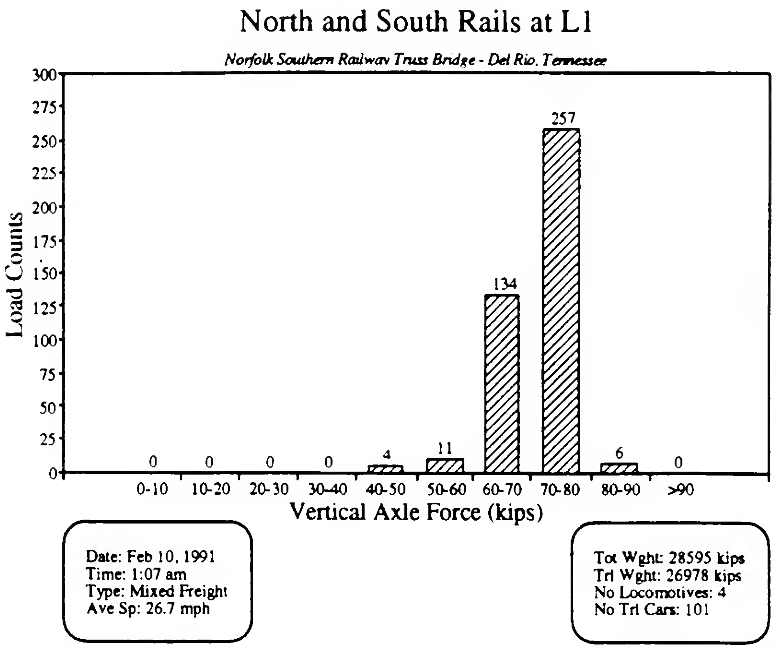


Exhibit 2.5. Histogram of Vertical Axle Loads at L1 for a Typical Train Crossing

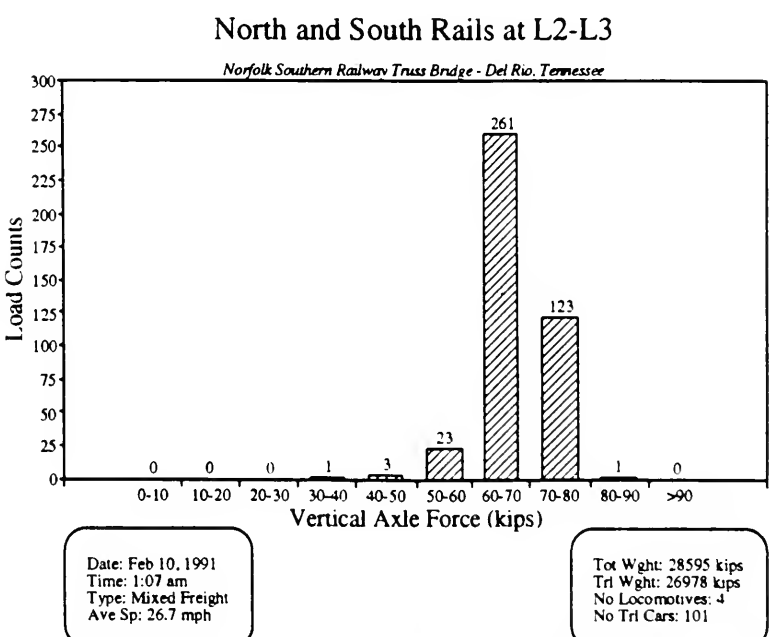
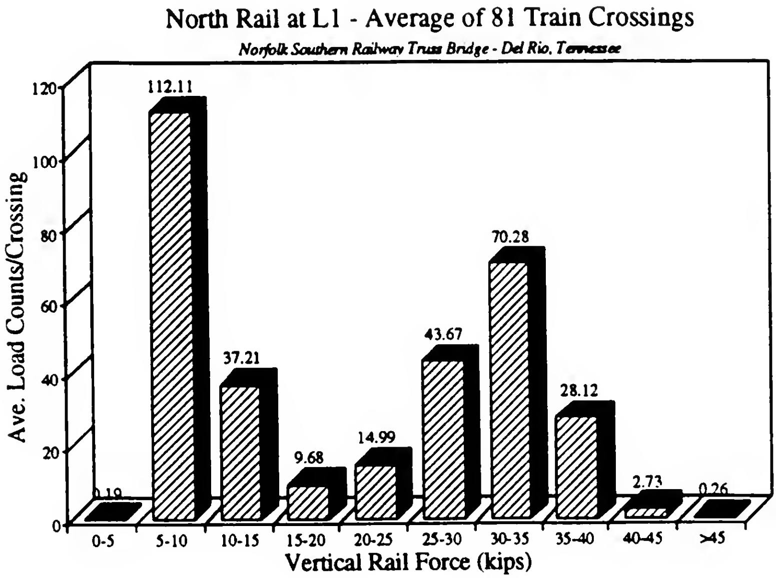


Exhibit 2.6. Histogram of Vertical Axle Loads at L2-L3 for a Typical Train Crossing



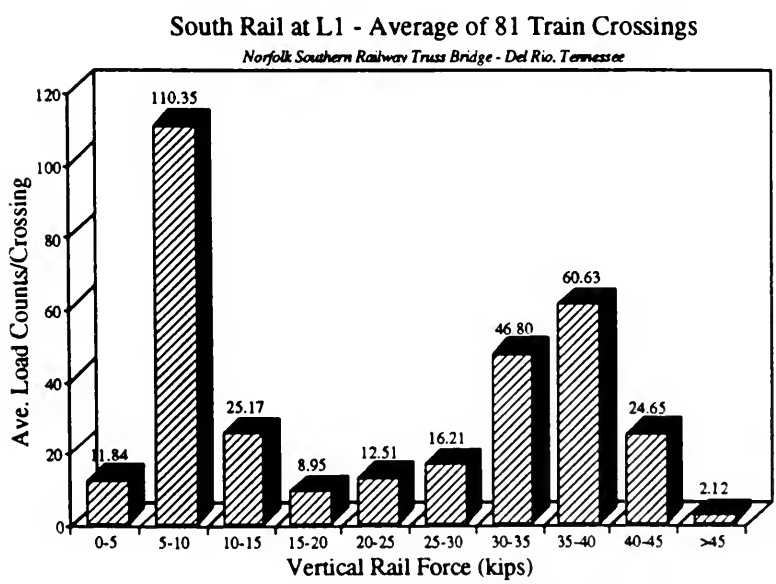
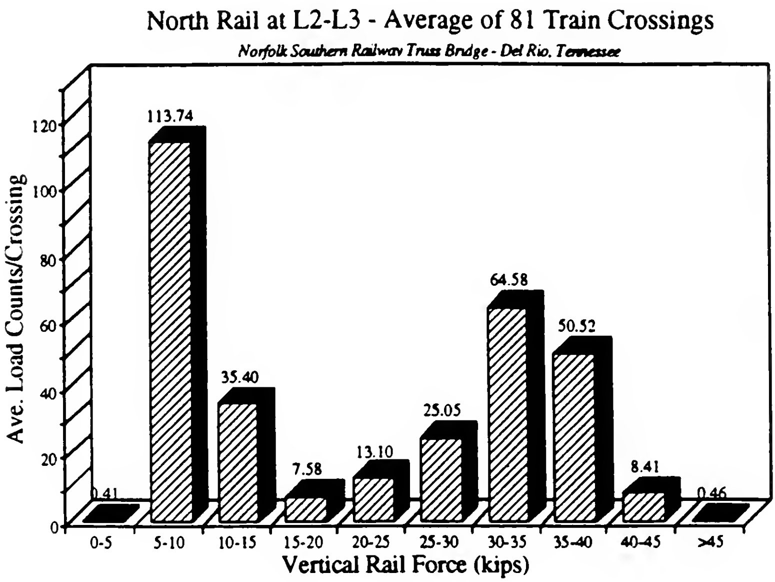


Exhibit 2.7. Histogram of Vertical Wheel Loads on the North and South Rails at L1 for 81 Train Crossings



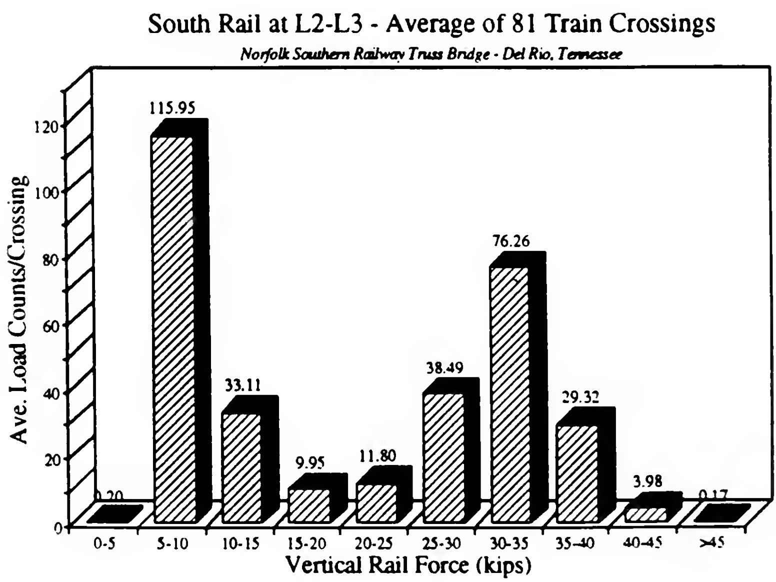
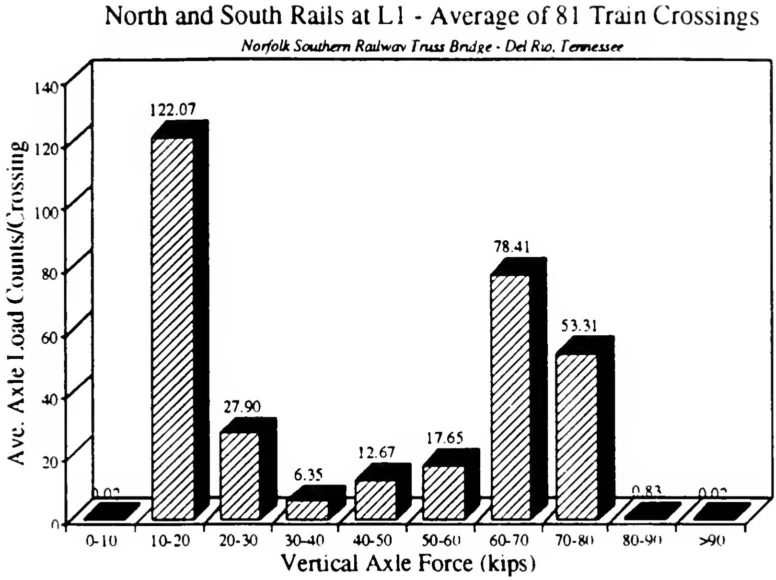


Exhibit 2.8. Histograms of Vertical Wheel Loads on the North and South Rails at L2-L3 for 81 Train Crossings



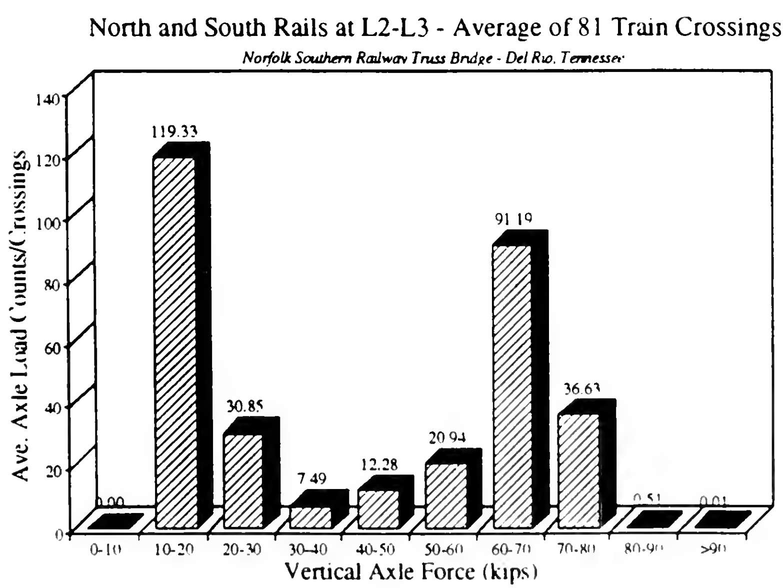


Exhibit 2.9. Histogram of Vertical Axle Loads at L1 and L2-L3 for 81 Train Crossings

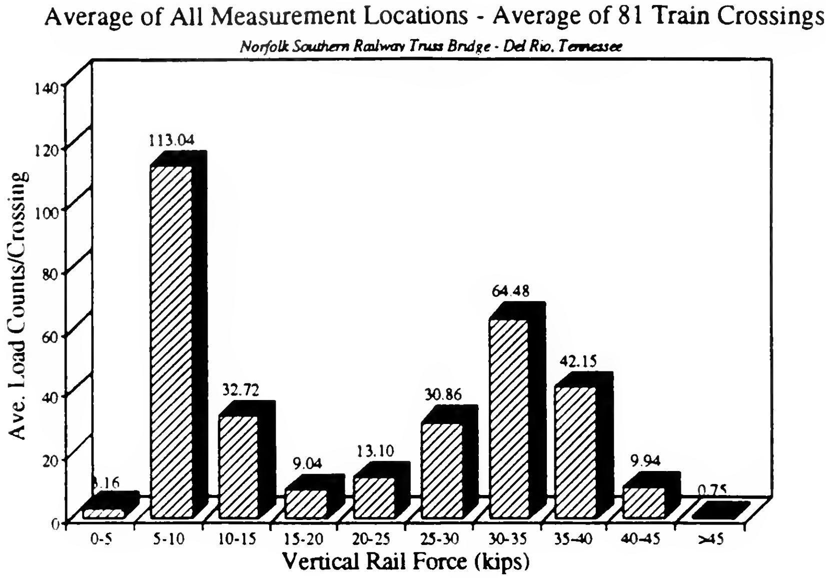


Exhibit 2.10. Histogram of Average Vertical Wheel Loads at the Site

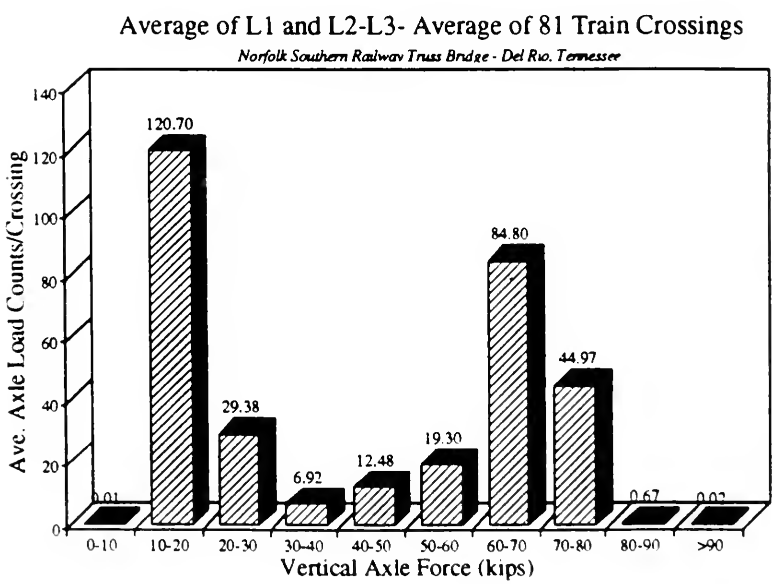


Exhibit 2.11. Histogram of Average Vertical Axle Loads at the Site

The distribution of vertical axle loads for locomotives is shown in Exhibit 2.12. Seventy three percent of the locomotive axle loads are between 60 and 70 kips and 24 percent are between 70 and 80 kips. This indicates that the average wheel loads for the locomotives are in the 30 to 40 kip range for 97 percent of the locomotives. About 60 percent of the locomotives had 6 axles and about 40 percent had 4 axles.

The histogram of vertical axle loads for freight cars is shown in Exhibit 2.13. This includes both ordinary freight cars and coal hopper cars. It is clear from this exhibit that the freight cars are the cause of the bimodal nature of the wheel load data for the site. Fourteen percent of the axle loads exceed 70 kips, but 40 percent are less than 20 kips (it should be noted that the coal cars are identified as freight cars in this study).

Data for the intermodal cars is given in Exhibit 2.14. These are the most lightly loaded cars of all. None of the axle loads measured were greater than 70 kips and only about one percent were in the 60-70 kip range.

The histograms of average axle loads is shown for the eastbound and westbound trains in Exhibit 2.15. This reflects the fact that a larger percentage of the traffic travels eastward from Knoxville to Ashville. The unit coal traffic is eastbound and the empties travel westbound.

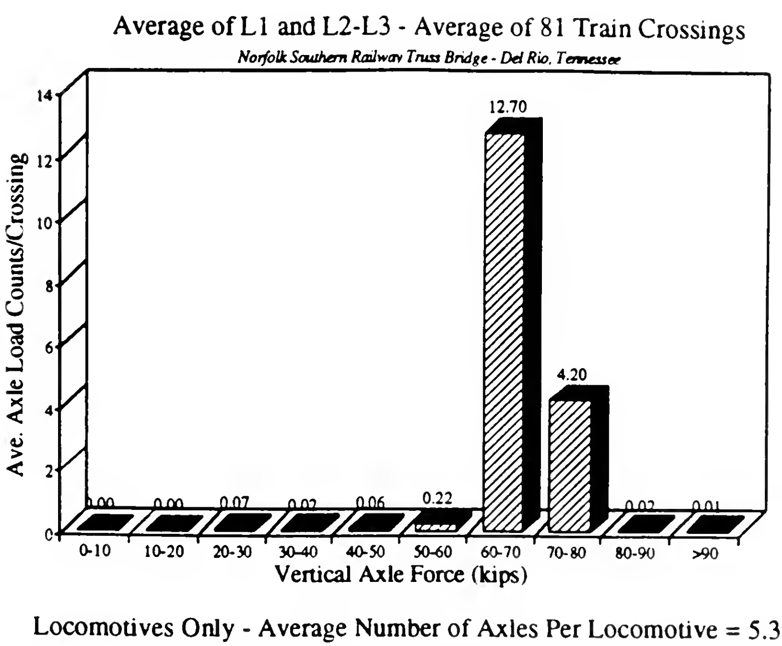
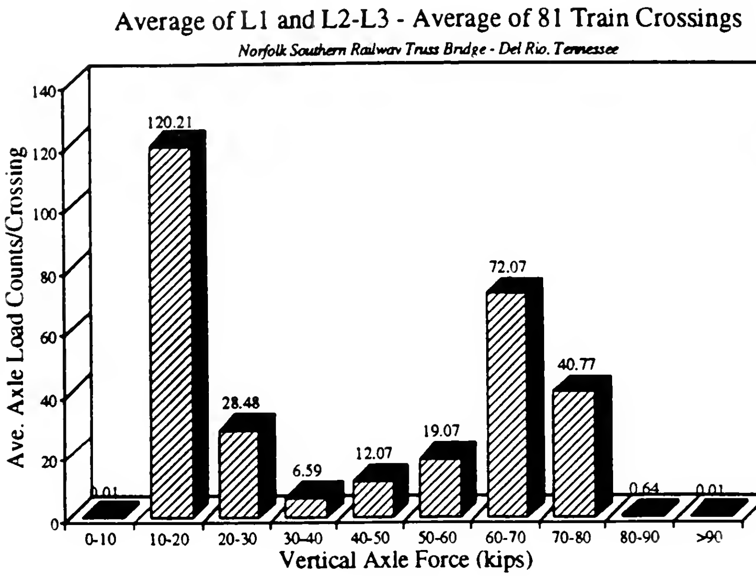
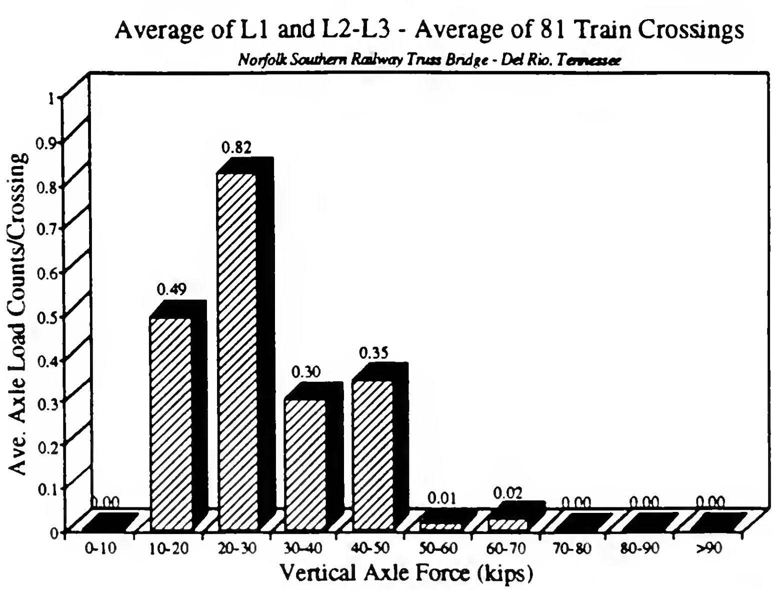


Exhibit 2.12. Histogram of Average Axle Loads at L1 and L2-L3 for Locomotives. Average for 81 Train Crossings.



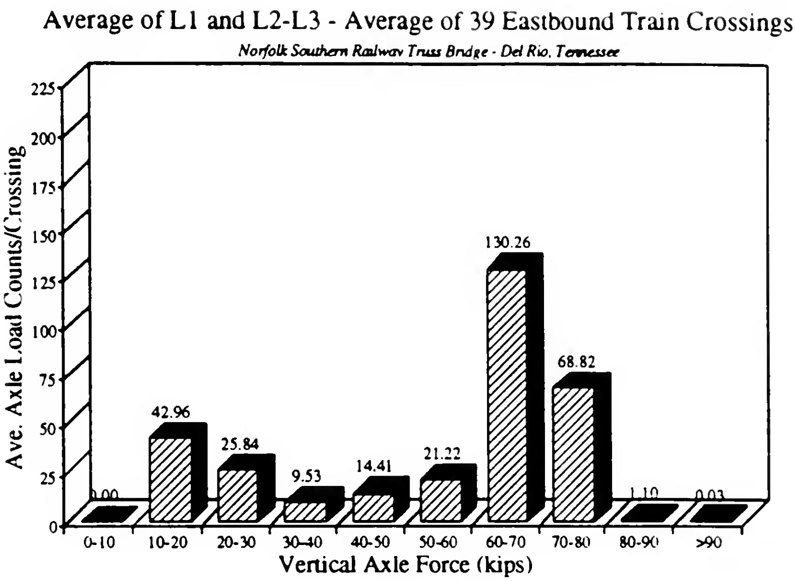
Freight Cars Only - Average Number of Axles Per Car = 4.00

Exhibit 2.13. Histogram of Average Axle Loads at L1 and L2-L3 for Freight Cars. Average for 81 Train Crossings.



Intermodal Cars Only - Average Number of Axles Per Car = 11.57

Exhibit 2.14. Histogram of Average Axle Loads at L1 and L2-L3 for Intermodal Cars. Average for 81 Train Crossings.



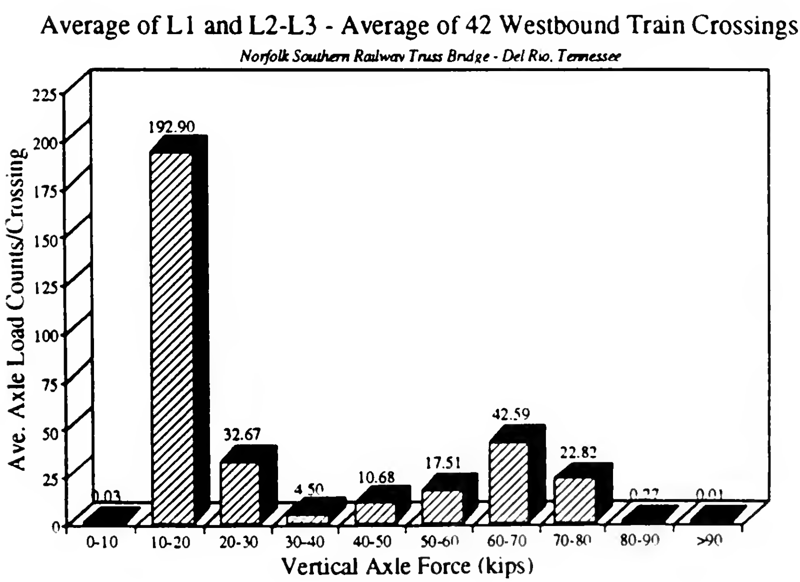


Exhibit 2.15. Histogram of Average Axle Loads at L1 and L2-L3 for Trains Travelling in the Eastward and Westward Directions

The lateral wheel forces were measured on both rails midway between L2 and L3. In addition to processing the outward and inward force on each rail as each wheel crossed the measurement point, the ratio of the lateral force to the vertical force was also calculated. The number of occurrences in each increment of load in the histograms is the average number in this increment for the north and south rails.

The average inward and outward lateral wheel loads for the north and south rails for the 81 train crossings are shown in Exhibit 2.16. The vast majority, 98 percent, of the lateral loads are less than four kips. Only one lateral wheel load in 405 exceeded 8 kips.

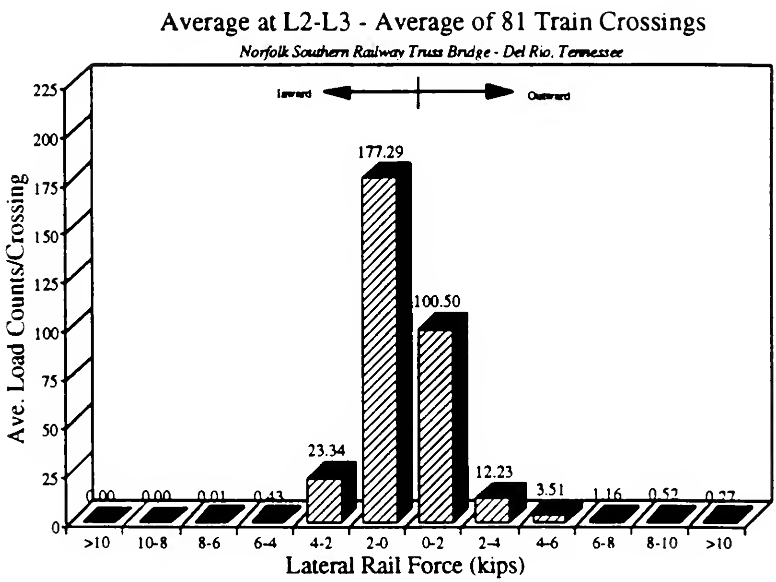


Exhibit 2.16. Histogram of Lateral Rail Forces at L2-L3. Average Counts at L2-L3 and Average of 81 Train Crossings.

The data for the ratio of lateral to vertical wheel load are shown in Exhibit 2.17. The data reveals that 99 percent of the L/V ratios are less than 0.2. About one wheel in 10,600 produces an L/V ratio greater than 0.5.

The measured stress time histories for a lower chord member and for an upper chord member during the first twenty seconds of a typical train crossing are shown in Exhibit 3.1. This is the same unit coal train whose wheel loads are shown in Exhibits 2.1 through 2.3. The member response of the bottom chord member is characterized by an initial large increase in stress as the bridge is loaded followed by a number of small stress cycles. The upper chord member has only the initial stress increase with essentially no additional cycles. The smaller cycles of stress in the lower chord are generally less than 1.0 ksi so they would not typically be counted since they would cause essentially no fatigue damage.

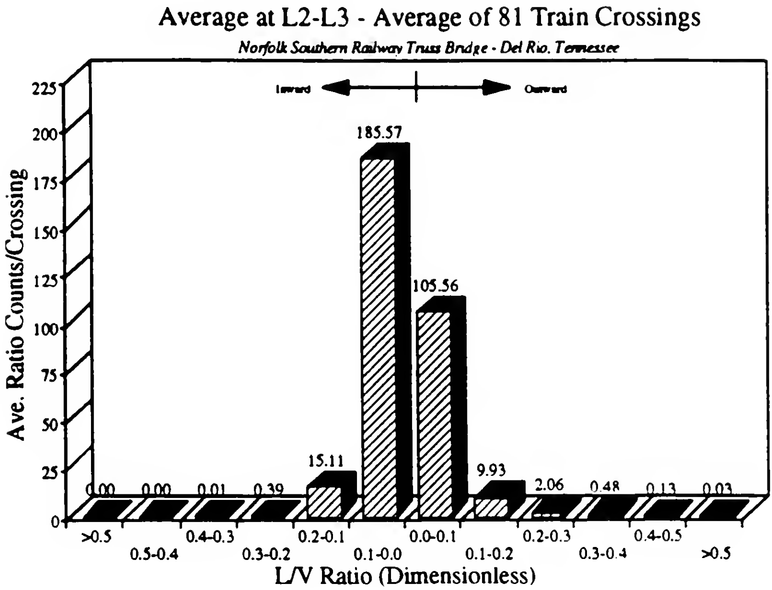


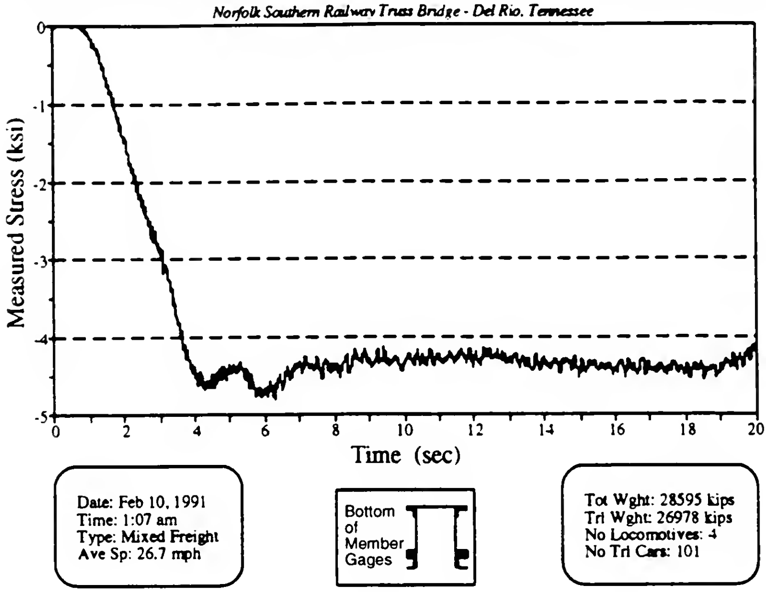
Exhibit 2.17. Histogram of Lateral/Vertical Force Ratio. Average Counts at L2-L3 and Average of 81 Train Crossings.

Histograms of the axial stress cycles for the lower chord members L0-L1, L1-L2 and L2-L3 for the north truss are shown in Exhibit 3.2. These are tabulated as the average number of occurrences per train crossing. Note that an occurrence rate of 0.00 means that no stress cycle was measured in that range. Also given on each histogram is the equivalent constant amplitude stress cycle, Sre, for the member and the number of occurrences. Data from the same three chord members in the south truss are not shown since they are essentially the same as for the north truss.

The maximum stress range encountered for L0-L1 and L2-L3 was in the 3.5-4.0 ksi range. The maximum for L1-L2 was in the 3.0-3.5 ksi range. The equivalent constant amplitude stress range for each is about 2.3 ksi with an average of about 3.3 cycles per train crossing. The histogram for L1-L2 measured near L2 is skewed because of the relatively larger number of small cycles measured there. This may have been influenced by additional noise in the channel or by localized vibration. The measurements for all other stress ranges are quite comparable to those measured at the other location. If only the cycles above 2.0 ksi are counted, all of the bottom chord members experience the equivalent of 1.5 cycles per train crossing at 2.9 ksi.

In-plane and out-of-plane bending moments will cause an increase in these stresses of 30 to 50 percent. Even so, the level of stress is such that fatigue damage, if occurring at all, should be occurring at a very low rate in the lower chord members under current operating conditions.

Measured Stress - Upper Chord U1-U2 Near U1 North Truss



Axial Stress - Lower Chord L1-L2 Near L2 North Truss

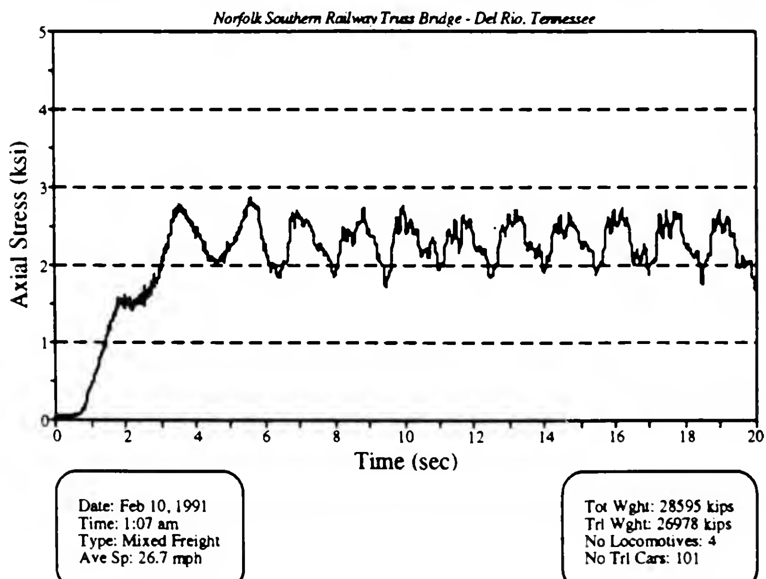


Exhibit 3.1. Time Histories of Chord Member Stresses During a Typical Train Crossing

Histograms of stress cycles per train crossing of the upper chord members are shown in Exhibit 3.3. Although these members experienced stresses as high as 5.5-6.0 ksi this is really insignificant since the members are in compression.

During previous tests, it was observed that the hangers at L1 experienced the largest stresses of any of the members. It was also pointed out by Steering Committee members that this is the member of a truss that most often experiences fatigue damage. As a result, most of the strain gages on the L1-U1 hangers were retained as active channels. The hangers are built-up I-shape members. One strain gage was placed near each edge of the flange (four gages) at three locations: near the bottom of L1-U1 for the north truss and near the top of L1-U1 for both trusses.

Typical time histories of stress for one flange edge near the bottom of L1-U1 and for one flange edge near the top of L1-U1 are shown in Exhibit 3.4. The response of the hanger is characterized by three modes of response. There is an initial large increase in stress caused by the initial loading by the train. On top of this is a cyclic stress response with a period of about 2.0 seconds and a stress range amplitude of about 4.5-5.0 ksi. This is the result of each pair of trucks crossing the bridge. Finally, there is a high frequency fluctuation of stress with an amplitude of about 1.0 ksi or less. This is the result of in-plane vibration of the hanger [2]. The maximum stresses are caused by the locomotives rather than the coal cars. However, the stress cycles from the coal cars are only slightly smaller than these. Thus, an increase in the allowable loads will have a significant impact on stress history of these hangers.

The histograms of stress cycles for each gage point for the three locations are shown in Exhibits 3.5 through 3.7. The data in Exhibit 3.5 indicates that the stress on the inside flange is higher than on the outside flange at the bottom of hanger L1-U1. The out-of-plane bending stress adds to the axial tension stress at these locations. The maximum stress experienced by the hanger is about 10.0 ksi. The equivalent constant amplitude stress for one point is 3.35 ksi with an average of 167 cycles per train crossing. This is only about one ksi higher than the level of stress experienced by the chords but it has significantly more cycles of occurrence. If only the stress cycles above 6.0 ksi are included, S_{re} = 8.9 ksi with 6.0 cycles of occurrence per train crossing. This is probably a more accurate portrayal of the level of stress in the member. This is definitely above the fatigue threshold level so fatigue damage is occurring in the hanger at this location.

The histogram of stress cycles per train crossing for the top of L1-U1 for the north truss are shown in Exhibit 3.6. Although an occasional stress range of 10.0 ksi is encountered, the equivalent constant amplitude stress range is only about 2.3 ksi at this location. There are considerably more cycles, however. The stress is probably lower at the top than at the bottom, because the out-of-plane moment is considerably lower at the top than at the bottom of the hanger. The number of cycles is probably higher at the top of the hanger because the in-plane moments due to vibration of the hanger are larger at the top. The data for the top of L1-U1 for the south truss, as shown in Exhibit 3.7, are similar to the data for the north truss. If only stress ranges above 6.0 ksi are counted for the OW point at the top of L1-U1 for the north truss, S_{re} = 7.7 ksi with 3.0 cycles of occurring, but at a fairly low rate.

The time histories of measured stress at one point near the bottom of diagonal U1-L2 of the north truss and at one point near the top are shown in Exhibit 3.8. The response of this member is similar to the hanger at L1-U1, but the cycles of stress that occur at a two-second period are only about 1.5 ksi peak to peak in amplitude. The maximum stress measured during the time period shown is about 6.8 ksi. The reason that the stress fluctuations are smaller for the diagonal than for the hanger is that the hanger responds only to axles that are loading the floor beam to which it is attached, whereas, the diagonal responds to some degree to all axles on the bridge.

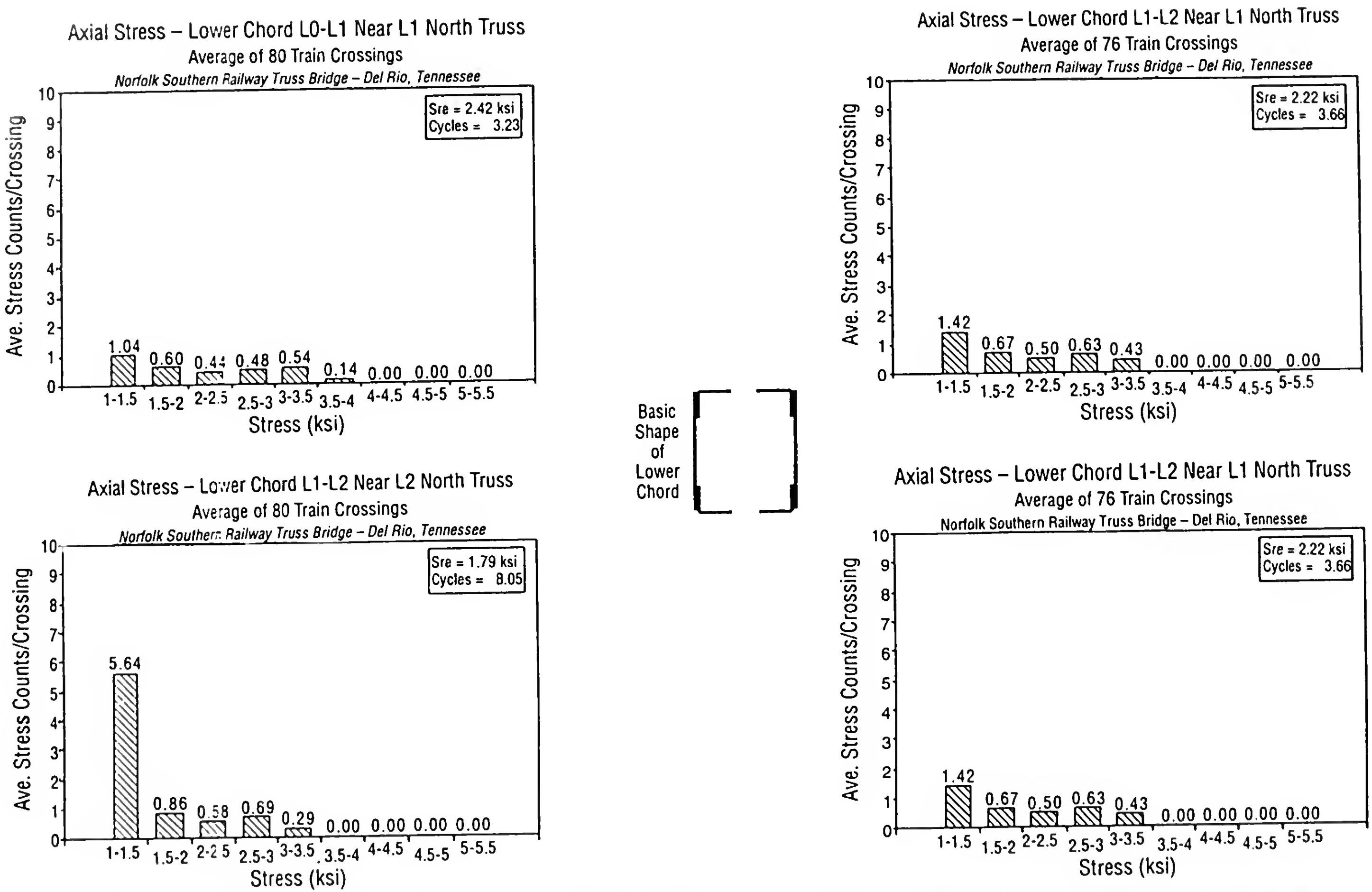
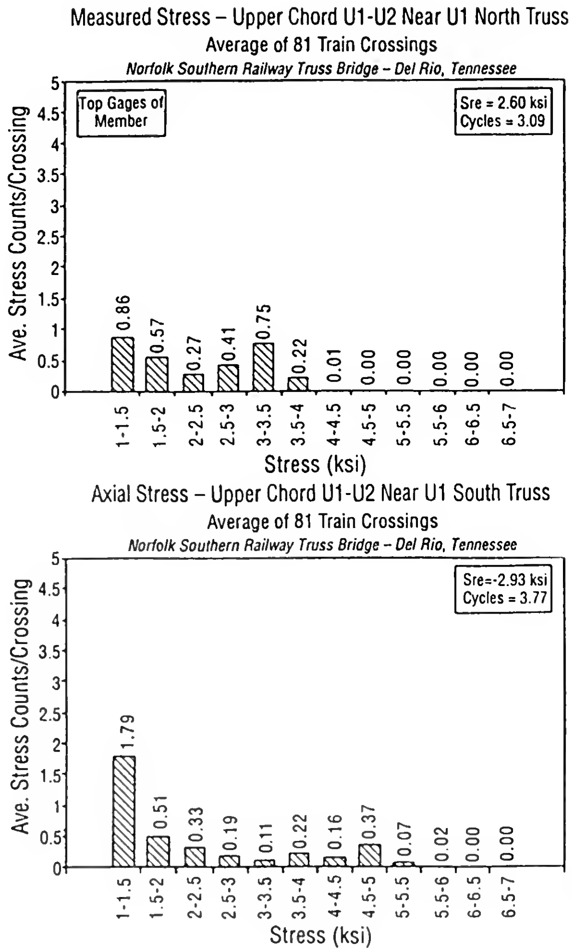
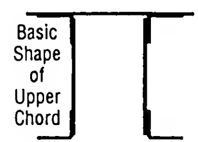


Exhibit 3.2. Histograms of Axial Stress in Lower Chord Members of the North Truss Given as Average Axial Stress Cycles Per Train Crossing





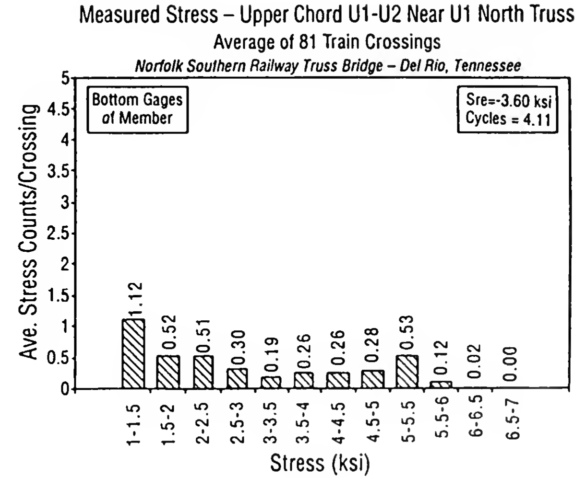
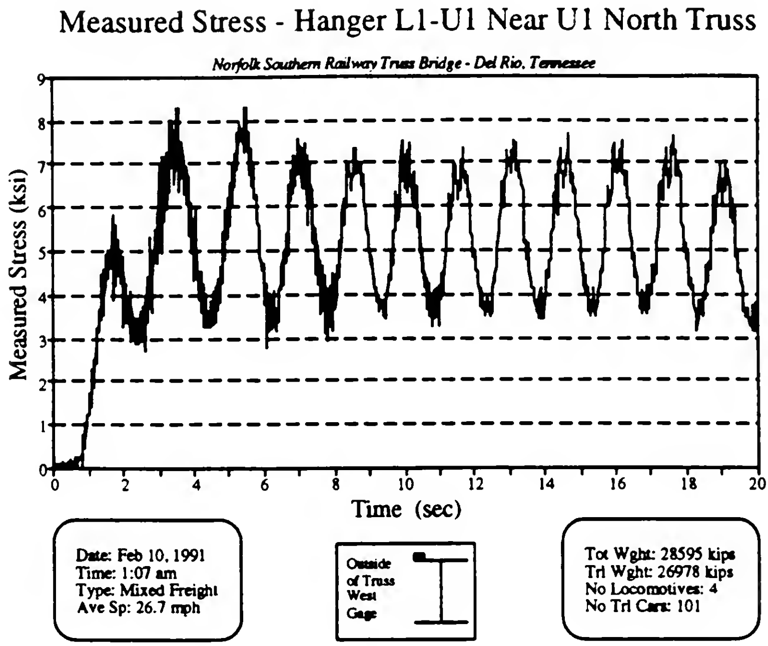


Exhibit 3.3. Histograms of Stress in Upper Chord Members Given as Average Axial Stress Cycles Per Train Crossing



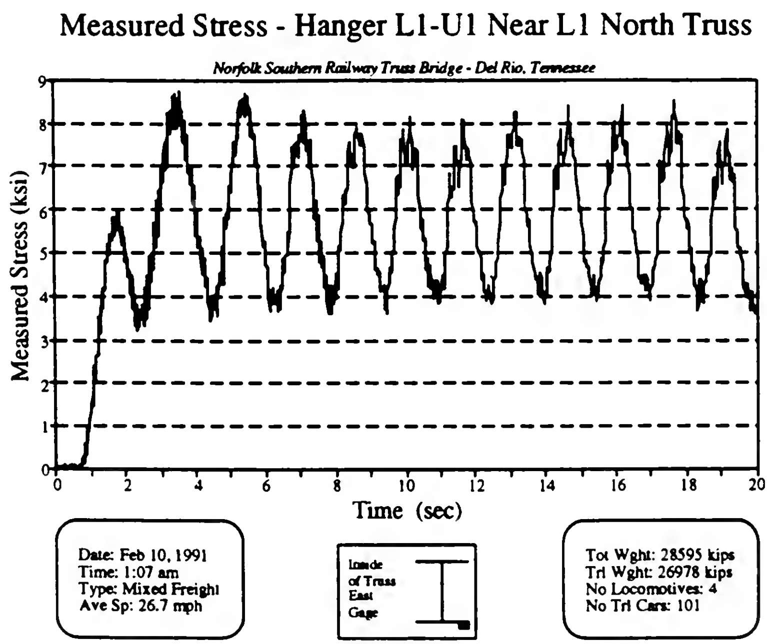


Exhibit 3.4. Time Histories of Measured Stress Near the Top of Hanger L1-U1 of the North Truss During a Typical Train Crossing

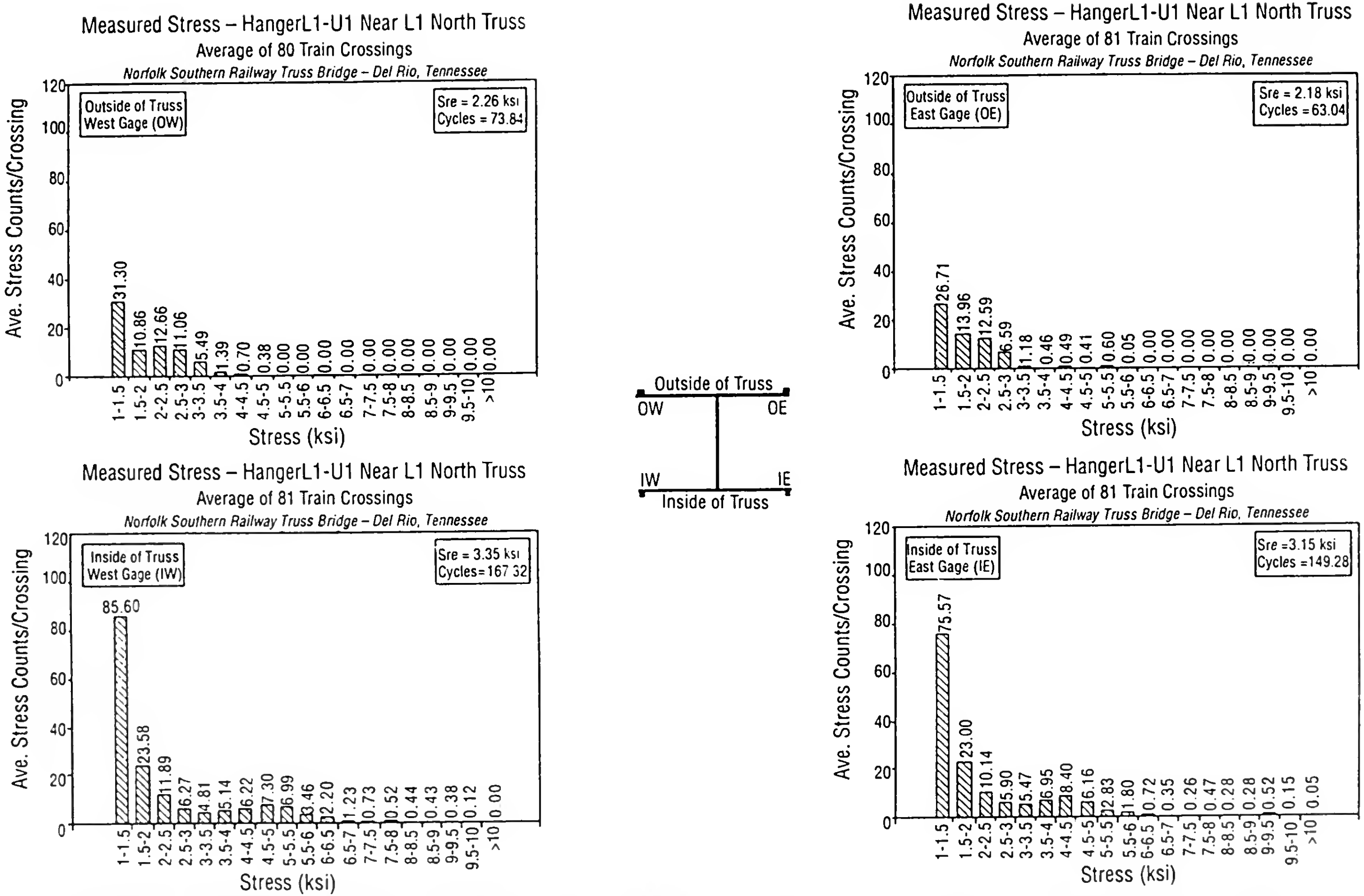


Exhibit 3.5. Histograms of Measured Stress Near the Bottom of Hanger L1-U1 of the North Truss Given as Stress Cycles Per Train Crossing

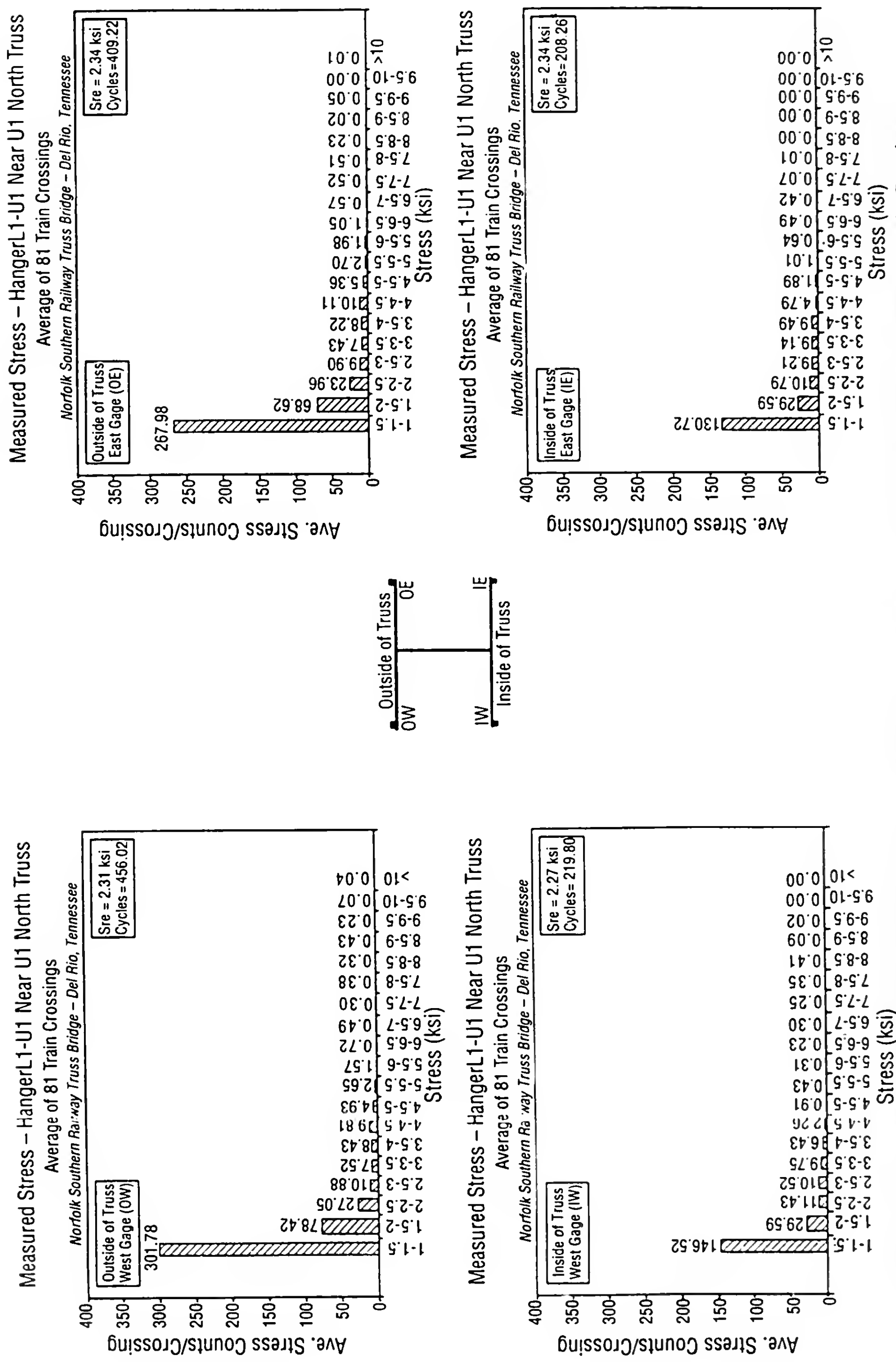


Exhibit 3.6. Histograms of Measured Stress Near the Top of Hanger L1-U1 of the North Truss Given as Stress Cycles Per Train Crossing

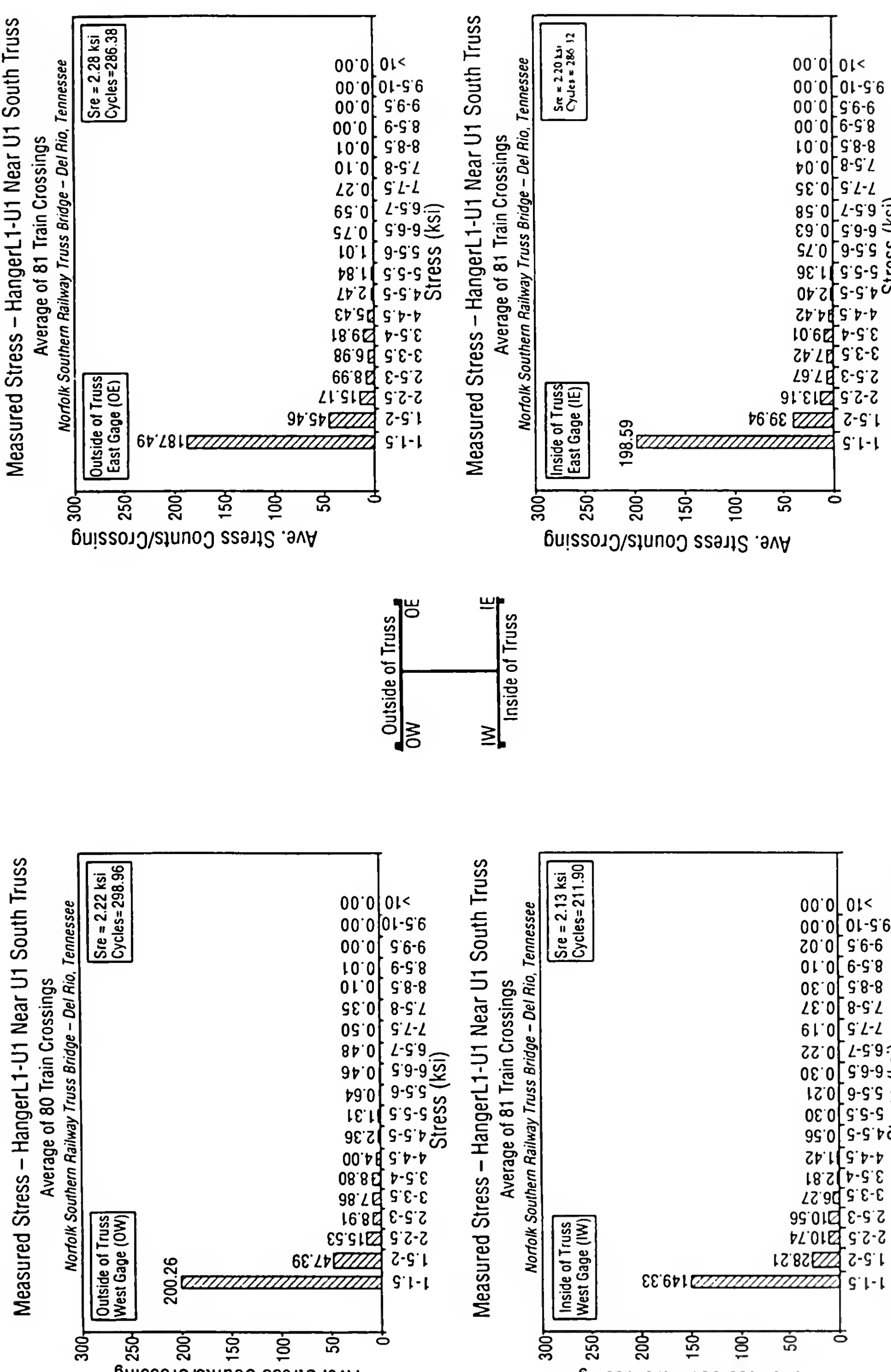
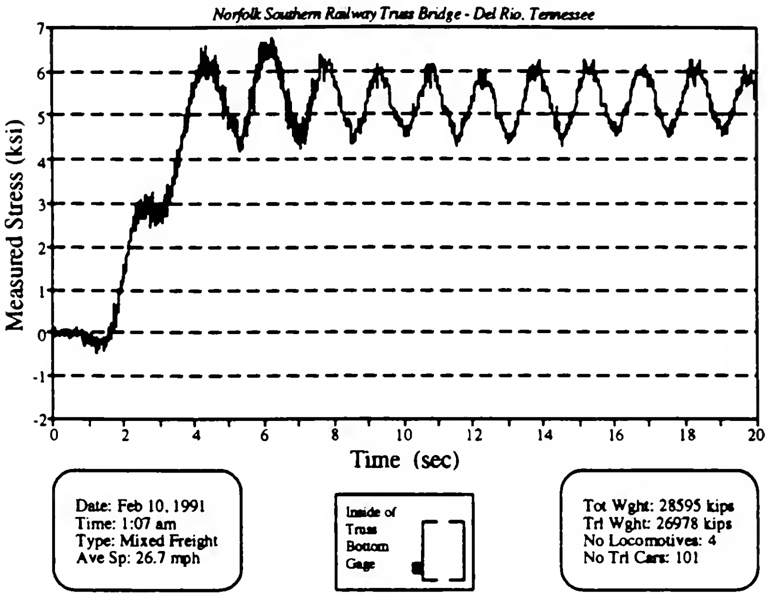


Exhibit 3.7. Histograms of Measured Stress Near the Top of Hanger L1-U1 of the South Truss Given as Stress Cycles Per Train Crossing

Measured Stress - Diagonal U1-L2 Near L2 North Truss



Measured Stress - Diagonal U1-L2 Near U1 North Truss

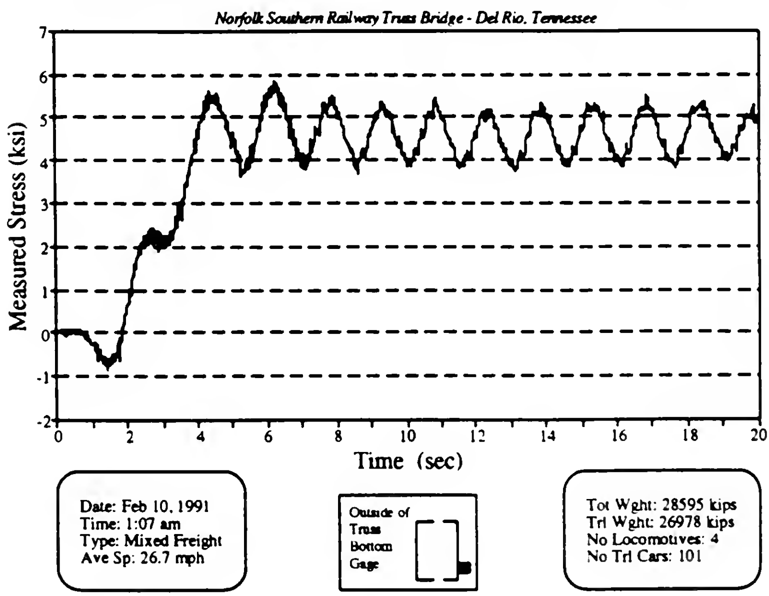


Exhibit 3.8. Time Histories of Measured Stress at Two Points on Diagonal U1-L2 of the North Truss for a Typical Train Crossing

Histograms of average stress cycles at four points near the bottom of diagonal U1-L2 of the north truss are shown in Exhibit 3.9. The two locations on the inside of the truss are more highly stressed than those on the outside of the truss. This indicates that out-of-plane bending is significant for this member near the bottom. The maximum stress reaches about 9.5 ksi at the worst location. The S_{re} values are about 2.4 ksi for points inside the truss and about 2.0 ksi for points outside the truss. If only stress cycles above 6.0 ksi are counted, S_{re} is 7.5 ksi at 1.4 cycles per train crossing.

The data for the upper location on this diagonal are shown in Exhibit 3.10. The data for the upper location of this diagonal in the south truss are shown in Exhibit 3.11. In both of these cases the locations on the bottom side of the diagonal are more highly stressed which indicates that in-plane bending is probably more significant near the top of the diagonal.

The maximum stress observed is about 9.0 ksi. The value of S_{re} is about 2.5 ksi at 35 cycles per train crossing for the entire histogram. If only cycles above 6.0 ksi are counted, S_{re} is 6.9 ksi at 1.0 cycle per train crossing. This indicates that the fatigue damage may be occurring under the current loading environment. The U1-L2 diagonal in the south truss has experienced a small fatigue crack near the top in the past that was observed and repaired.

The time histories of bending stress near an end of the north and south stringers between L2 and L3 are shown in Exhibit 3.12. The fact that these oscillate between positive and negative stress indicate that the stringers are acting as continuous beams supported at each floor beam. The tests revealed that the stringers have almost 100 percent fixity at their ends [1,2].

Histograms of average stress ranges per train crossing are shown in Exhibit 3.13. The maximum location from a fatigue standpoint is the L3 end of the L2-L3 stringer on the south side. The maximum bending stress is about 6.0 ksi and S_{re} is 3.35 ksi at 83 cycles per train crossing. Unfortunately, the center moment was not measured during these tests. During the dynamic part of the tests it was observed that the moments at mid-span were about the same as those at the ends.

The time histories of the bending stresses near the ends of floor beams L1 and L2 are shown in Exhibit 3.14. These are quite regular but also quite small. Histograms of stress cycles are shown in Exhibit 3.15. These figures indicate a very low S_{re} and a low number of counts per train crossing at each location. The tests revealed that the center moments of the floor beams were four to five times larger than the end moments because there was very little end fixity for the floor beams. If this relationship held for these tests, the S_{re} value would be between 5 and 6 ksi and beams were four to five times larger than the end moments because there was very little end fixity for the floor beams. If this relationship held for these tests, the S_{re} value would be between 5 and 6 ksi and there would be many cycles per train crossing. This would be very significant from a fatigue standpoint.

The AREA Manual for Railway Engineering [3], Chapter 15, recognizes the use of measured stress cycles under revenue traffic as an effective means of evaluating the effects of train traffic on the fatigue resistance of existing bridges. The estimated number of cycles to failure for a riveted member is given as

$$N = A \times S_R^{-3} \qquad (1)$$

where

N = estimated minimum number of cycles to failure

S_R = allowable stress range, ksi

A = 4.4×10^9 for Category C; 2.2×10^9 for Category D

If the rivets can be shown to be tight, Category C is appropriate. However, if the rivets are loose, Category D is appropriate. If at least 0.1 percent of the cycles are above the fatigue threshold level of 7.0 ksi (Category D), then the line represented by Equation 1 should be extended indefinitely. The value of S_{re} calculated from histograms of measured stresses may be substituted for S_R in Equation 1.

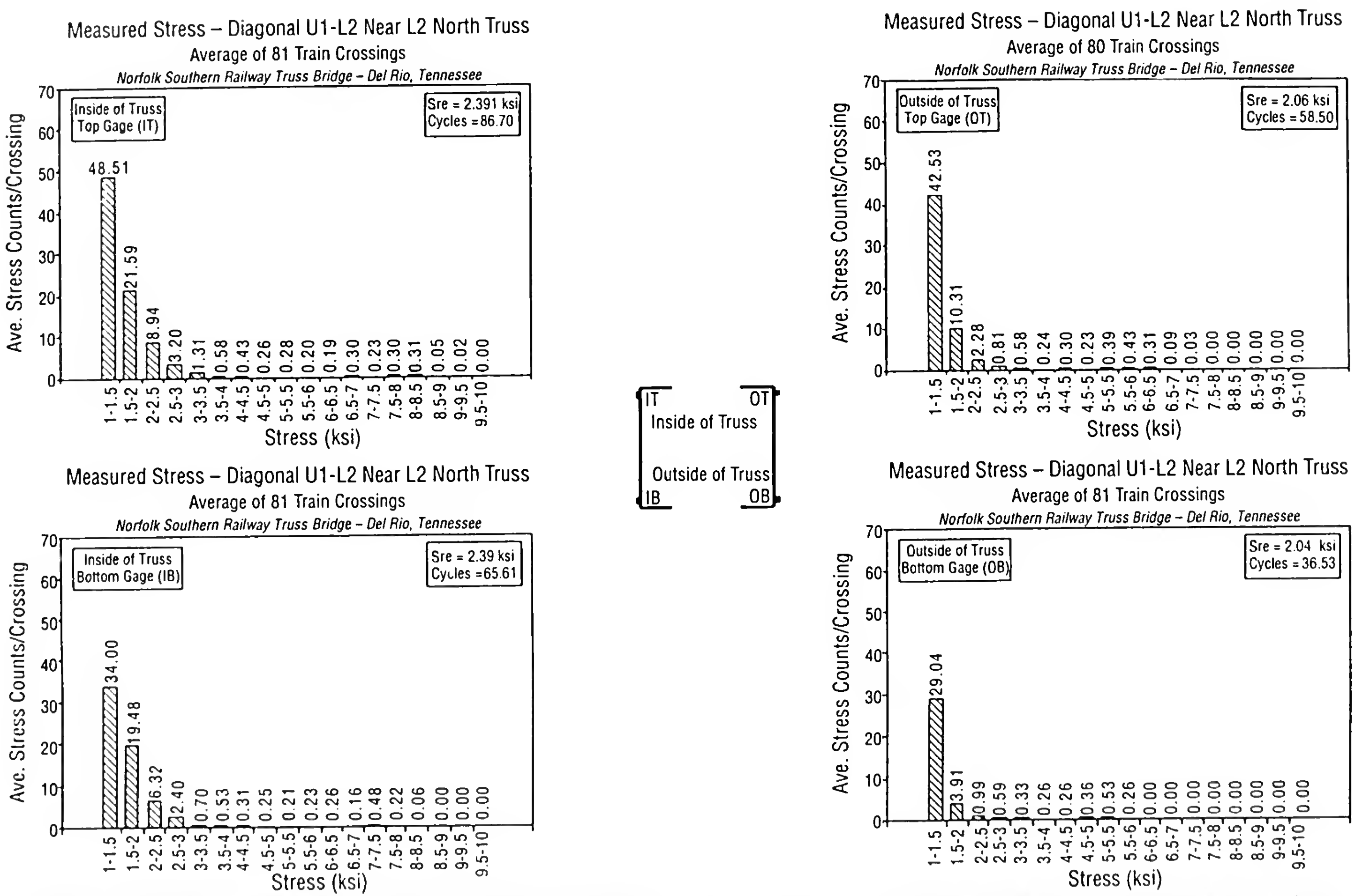


Exhibit 3.9. Histograms of Measured Stress Near the Bottom of Diagonal U1-L2 of the North Truss Given as Average Stress Cycles Per Train Crossing

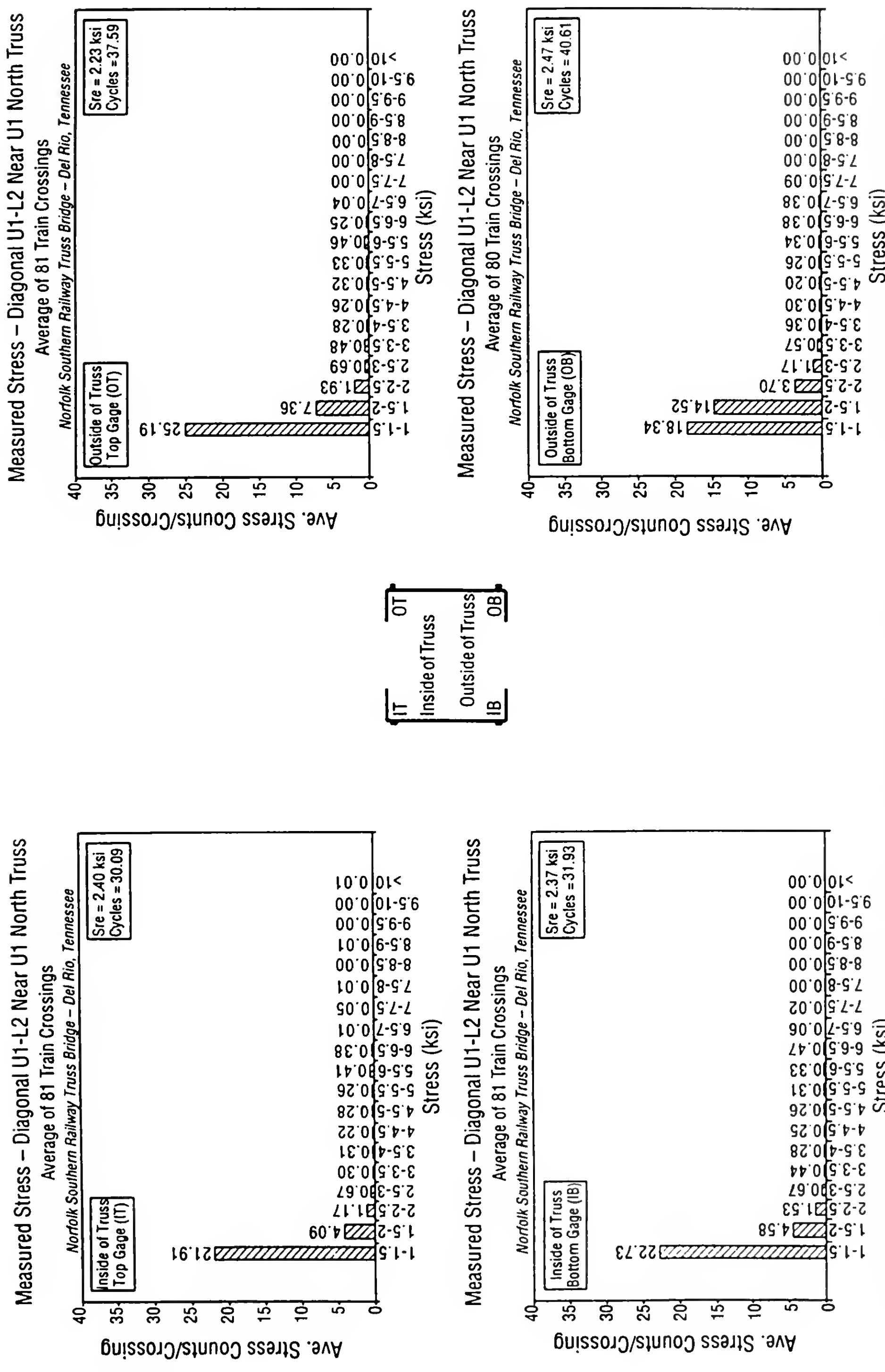


Exhibit 3.10. Histograms of Measured Stress Near the Top of Diagonal U1-L2 of the North Truss Given as Average Stress Cycles Per Train Crossing

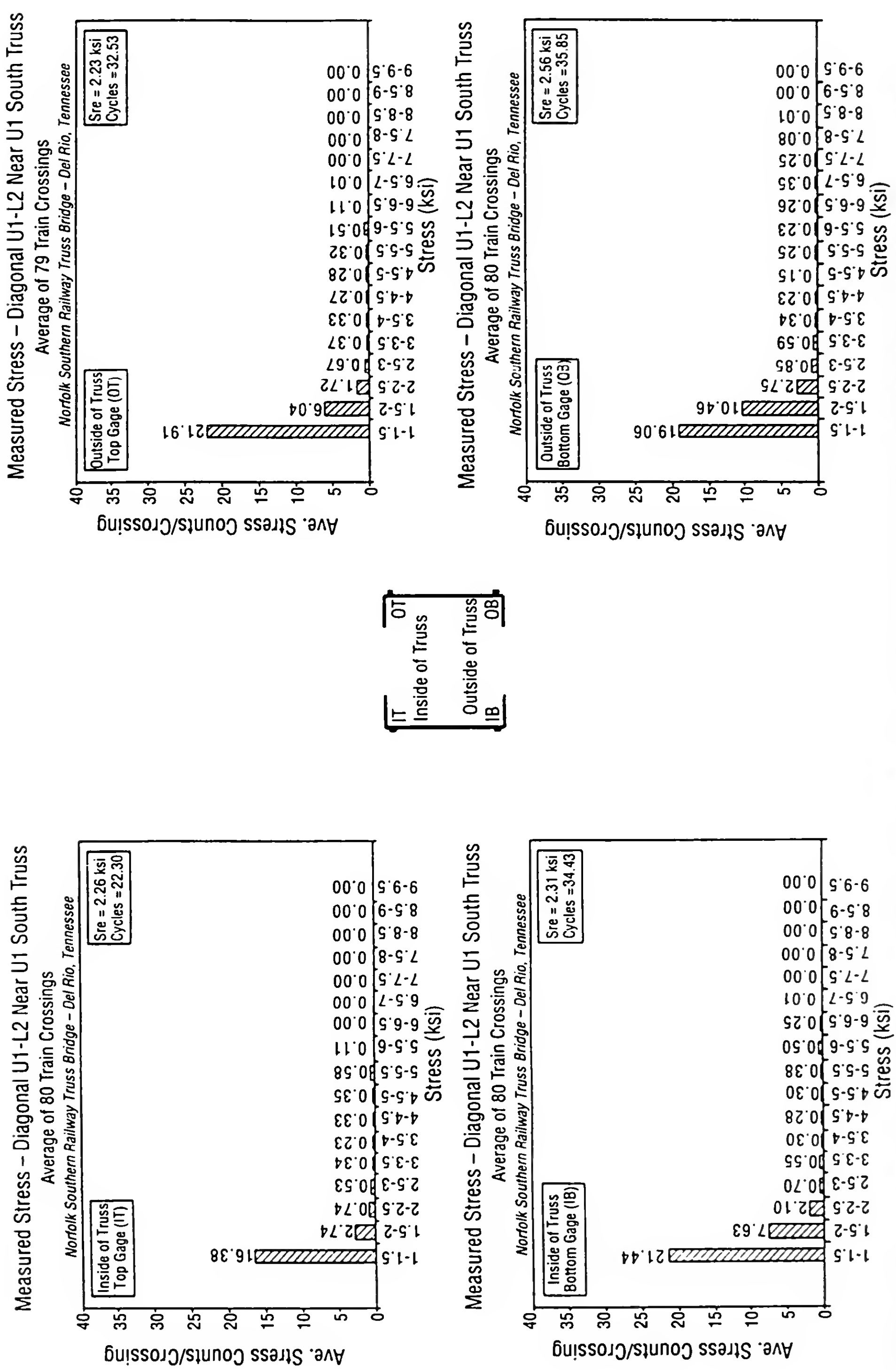
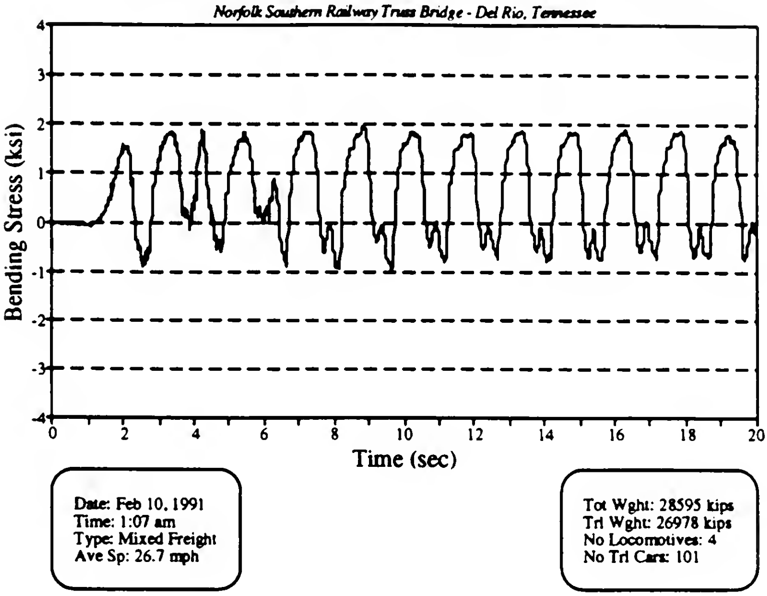


Exhibit 3.11. Histograms of Measured Stress Near the Top of Diagonal U1-L2 of the South Truss Given as Average Stress Cycles Per Train Crossing

Bending Stress - North Stringer L2-L3 Near L2



Bending Stress - South Stringer L2-L3 Near L3

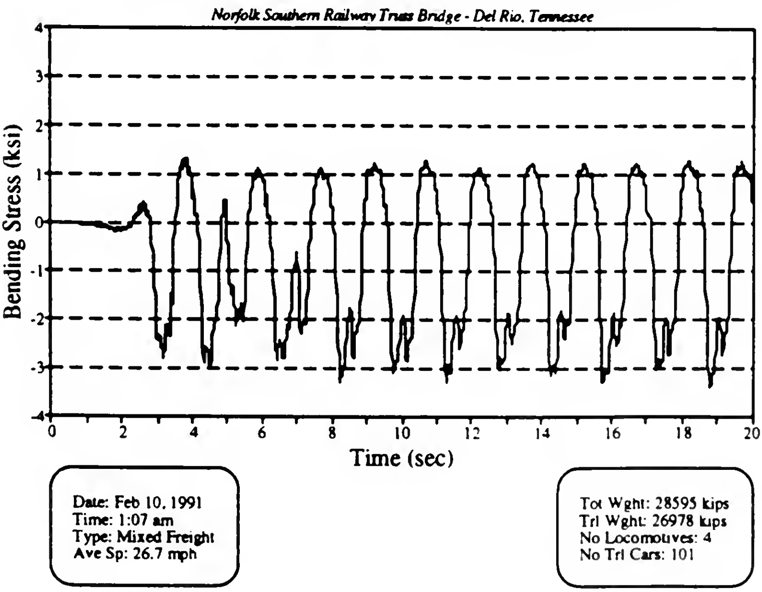


Exhibit 3.12. Time Histories of Bending Stress at the End of the North and South Stringer L2-L3 During a Typical Train Crossing

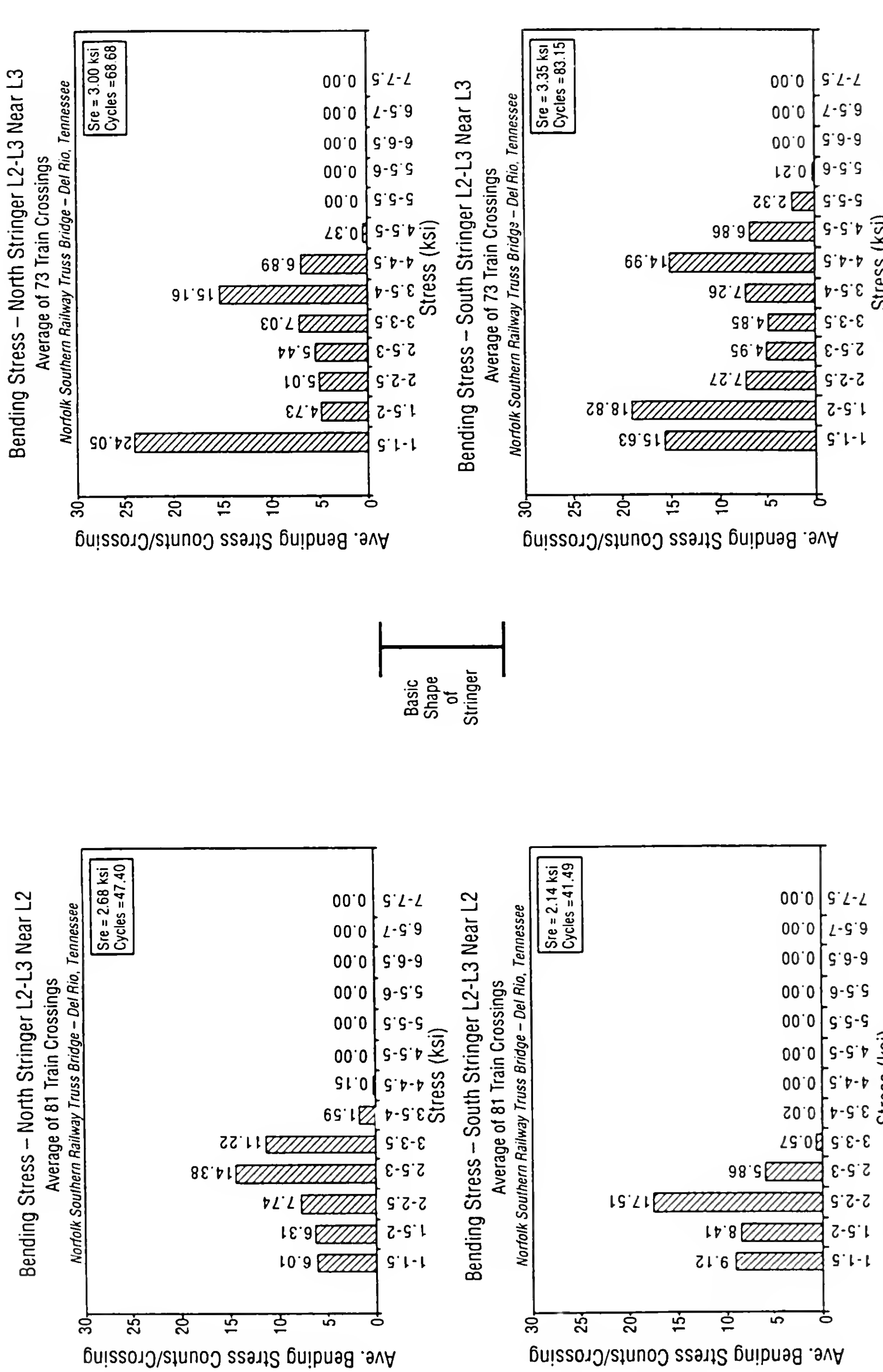
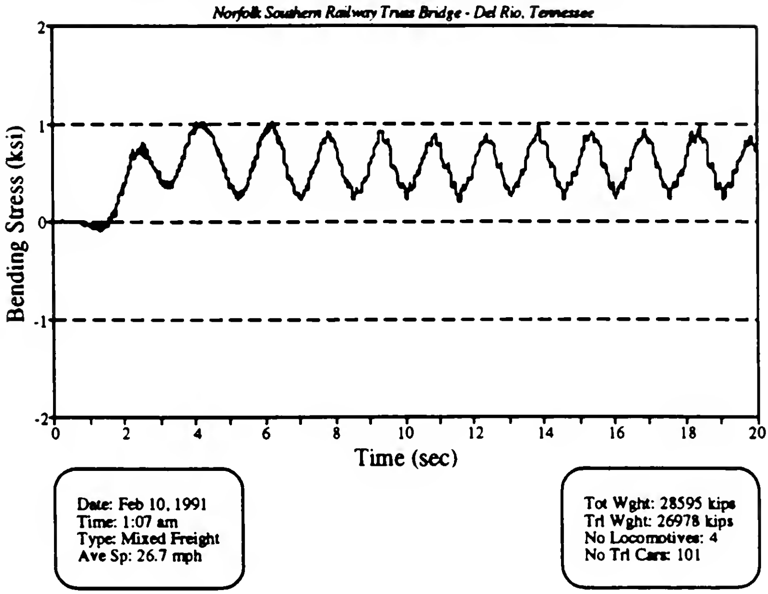


Exhibit 3.13. Histograms of Bending Stress Ranges for Selected Stringers Given as Average Bending Stress Counts Per Train Crossing

Bending Stress - Floor Beam at L2 Near South End



Bending Stress - Floor Beam at L1 Near South End

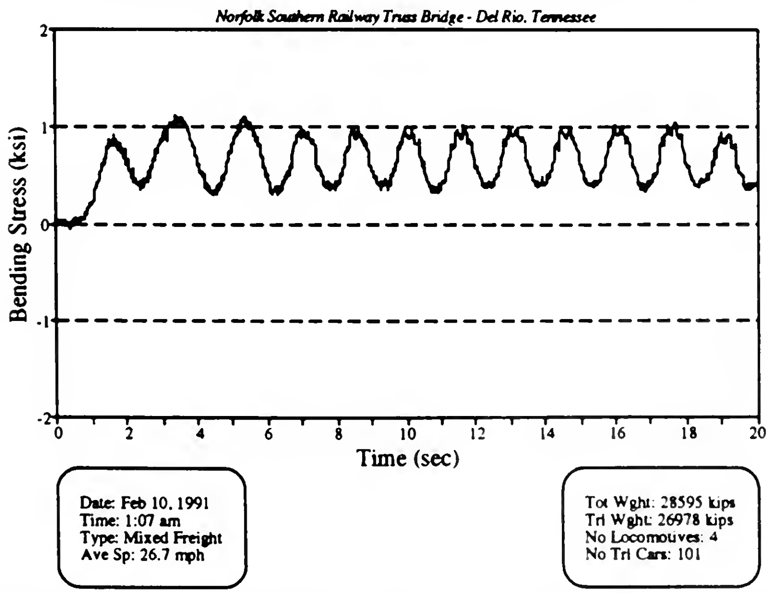


Exhibit 3.14. Time Histories of Bending Stress at the End of Floor Beams L1 and L2 for a Typical Train Crossing

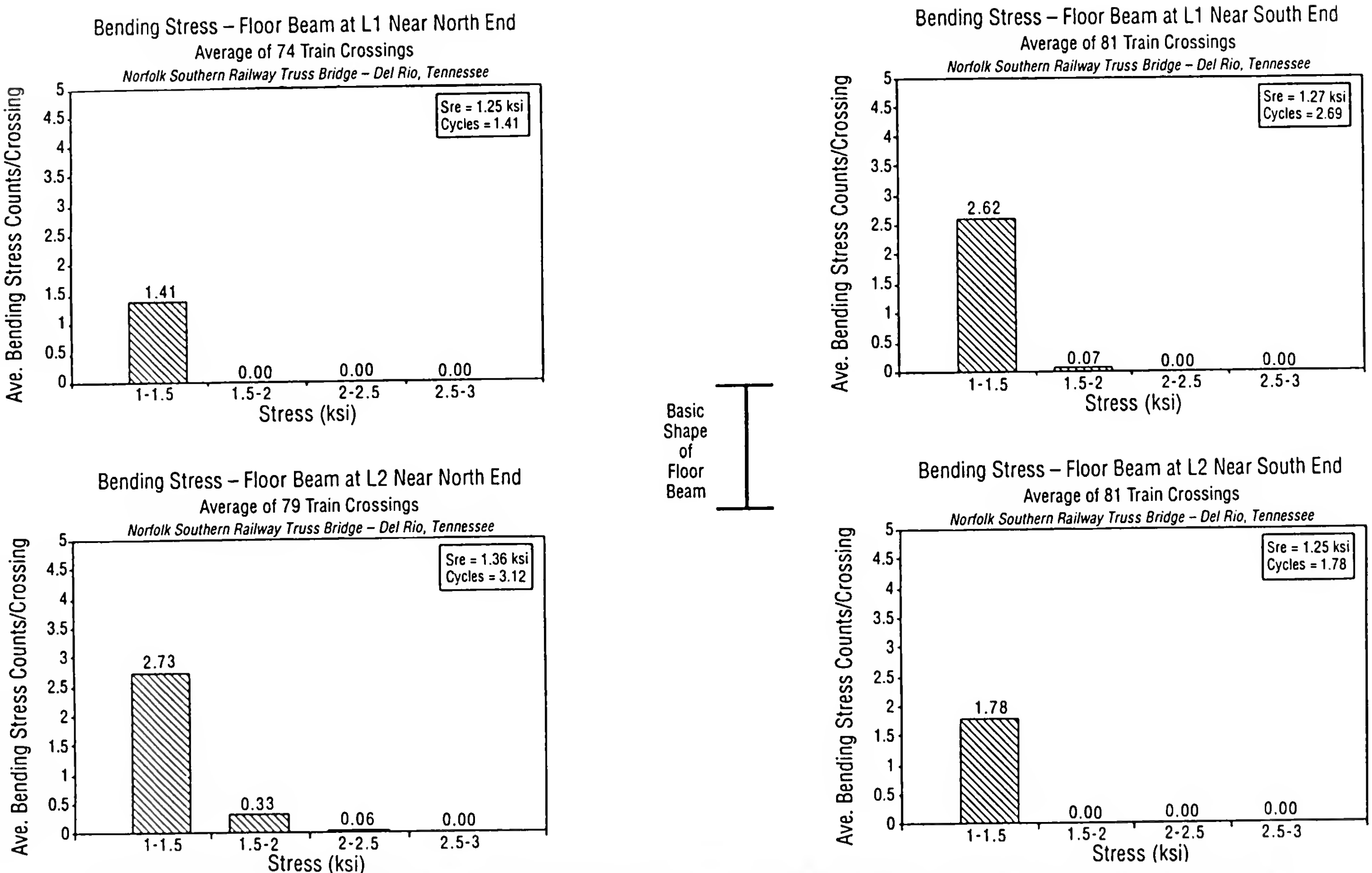


Exhibit 3.15 Histograms of Bending Stress Ranges for Selected Floor Beams Given as Average Bending Stress Counts Per Train Crossing

In the discussion of the data for each member type, two values of S_{re} were given with the appropriate number of cycles for each. One case included all stress ranges and the second included only those above a certain stress range. At first glance, it appears that the combination that included the highest S_{re} would be the governing one. This is not the case for this data, however. Using Equation 1 and the data for the hanger histograms, it can be shown that the maximum situation is the one that includes all of the stress ranges down to 1.0-1.5 ksi. The reason for this is that the number of cycles increases dramatically at these lower stress ranges. This more than compensates for the lower value of S_{re}.

A fatigue evaluation for seven member locations is given in Exhibit 3.16. The hangers appear to be the members undergoing the greatest amount of fatigue damage. The worst location is near the bottom of hanger L1-U1, where 700,000 train crossings (assuming Category C) under current traffic operations would be expected to cause failure of a new member. If we assume that 10 trains per day cross the bridge (this includes empties), then it would take 192 years for failure to occur in a new member. It would take 96 years to fail if Category D is appropriate.

A proper estimate of the remaining life of the member would require checking the yearly traffic on the bridge throughout its life and estimating the percent of the fatigue life remaining at this time. Then, by projecting the future traffic or by assuming that the loading magnitude and mix will remain the same, the remaining years of service could be estimated. In general the damage done in the early years of life will be so small that it can be neglected. Based on the data given in Exhibit 3.16, it does not appear that a problem exists for this bridge in the near future.

In a companion study, Monac International Corporation performed acoustic emission tests at eight locations on various bridge members. As a fatigue crack propagates under load, it emits a very high frequency sound. Using special transducers and a patented pattern recognition system, Monac can estimate the crack growth rate if a crack is growing. The eight locations that were checked were as follows: (1) diagonal U1-L2 near U1 inside south truss; (2) diagonal U1-L2 near U1 inside north truss; (3) hanger L1-U1 near L1 outside north truss; (4) hanger L1-U1 near L1 inside north truss; (5) end of floor beam at L1; (6) end of stringer L2-L3; (7) hanger L1-U1 near U1 inside south truss; (8) hanger L1-U1 near U1 outside south truss.

The test results indicated that crack growth was present at locations 2 and 7. However, the estimated growth rate was very small and indicated that the cracks were in the initiation stage. No crack growth was detected at the other locations. The conclusion was that fatigue is not a problem based on the eight locations that were tested which is consistent with the measured strain data.

One should be very careful about making a detailed point by point comparison of the two test methods. Due to the low level of strain in the members, it is questionable whether the AREA method is appropriate at many of the locations. A large extrapolation from available test data is required. It would be very interesting and informative to compare the two test methods using a bridge where member stresses are a lot higher and where fatigue damage is expected or is known to be present.

The measured data and analytical studies lead to the following conclusions:

1. Vertical and lateral wheel loads and member strains may be measured accurately for bridges under revenue traffic with minimal instrumentation requirements.
2. The average axle loads on this line are relatively small, but this is due primarily to the presence of empty coal trains and lightly loaded intermodal traffic.
3. Although a small percentage of the axle loads exceed 80 kips, these do not have a significant effect on the strains that develop in the bridge members. These are caused by dynamic response of the cars uneven loading or poor suspension behavior, so other axles contributing to the strain in a particular member have correspondingly smaller loads. The net result is that the stress in the members tends to increase or decrease based on the average axle loads of adjacent trucks rather than individual axle loads.

Member and Place	Location on Member	Sre (ksi)	Cycles Per Train Crossing	Total Cycles to Failure		Train Crossings to Failure	
				Category C (x 10^6)	Category D (x 10^6)	Category C (x 10^5)	Category D (x 10^5)
Hanger L1-U1 Near L1 North Truss	IW	3.35	167.32	117	58	7.00	3.50
Hanger L1-U1 Near U1 North Truss	OE	2.34	409.22	344	172	8.41	4.21
Hanger L1-U1 Near U1 South Truss	OE	2.28	286.38	372	186	13.00	6.50
Diagonal U1-L2 Near L2 North Truss	IB	2.39	65.61	320	160	48.77	24.39
Diagonal U1-L2 Near U1 North Truss	OB	2.47	40.61	293	147	72.21	36.11
Diagonal U1-L2 Near U1 South Truss	OB	2.56	35.85	262	131	73.08	36.54
South Stringer L2-L3 Near L3	End Bending	3.35	83.15	117	59	14.07	7.04

Exhibit 3.16. Table of Fatigue Data for Selected Members

4. The average stresses are small in all of the members that were checked. Fatigue damage in members where it is occurring at all, should be occurring at a very low rate. The average stresses per train crossing are somewhat smaller than expected because of the empty train crossings.
5. The combination of stress measurements and analytical models available in the most recent AREA manual provide a convenient method for evaluating the remaining life of existing bridges.
6. Localized vibration of members during a train crossing leads to a large number of cycles of member response at low stress levels. These cycles were found to be significant for the level of loads that the bridge is currently carrying. If the wheel loads are increased significantly, it is not known if these vibration induced stresses will also increase. If not, then they will be of less significance as the average member stress increases.
7. Although the stresses are accurately predicted by a three dimensional rigid frame model and static analysis, the number of cycles of member stress per train crossing is not. This is because of the member vibration. Therefore, field measurements of stress in selected members is required for truss bridges if confidence is to be placed on predictions of remaining life.
8. Another uncertainty in the prediction of remaining life is the use of current fatigue damage models where measured Sre stresses are below about 7.0 ksi. The current data base has very little data for low stress reversals and large number of cycles. There is currently no good agreement among experts on the accuracy of current models in this region. Therefore, it is recommended that additional tests of full-size riveted members be conducted to fill this void in the data base.
9. A truss that develops considerably larger stresses under load should be tested using strain gage technology and using acoustic emission methods so that a better comparison and evaluation of the two test methods may be made.
10. The specific results from these tests should not be applied to other truss bridges with different loading environments.

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1. Sharma, V., J. Choros, and A. J. Reinschmidt. "Static and Dynamic Testing of a Through Truss Bridge," Association of American Railroads, Report No. R-806, AAR Research Center, Chicago, IL (March 1992).
2. Tobias, D. H., D. A. Foutch, and J. Choros. "Investigation of an Open Deck Through-Truss Railway Bridge: Work Train Tests," Association of American Railroads, Report No. R-828, AAR Research Center, Chicago, IL (October 1992).
3. AREA. "Chapter 15," Manual for Railway Engineering, American Railway Engineering Association, Washington, D.C. (1992).

THE NEW HIGH SPEED LINE IN THE NORTH OF FRANCE (TGV-NORD)

by Jean-Pierre Pronost*

I. INTRODUCTION

I.1. The network of new high-speed lines in service now in French Railways has a total length of 630 miles. It consists of three lines (Fig. 1):

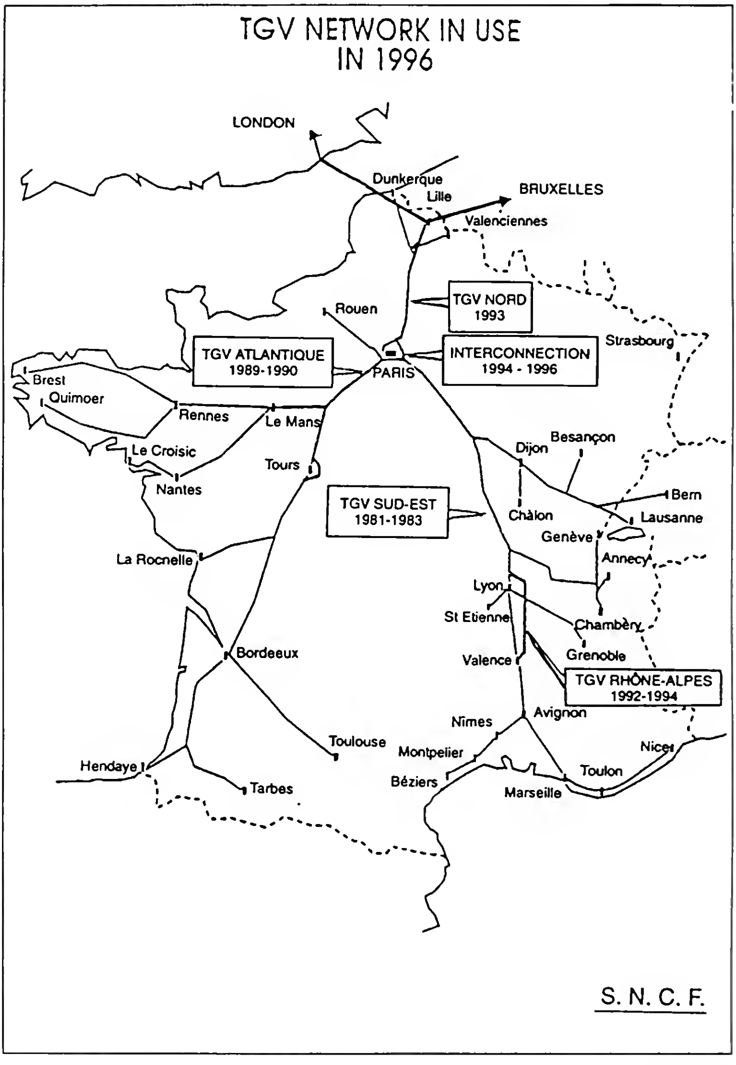


Figure I

*Directeur de la Ligne Nouvelle du TGV-Nord at de l'Interconnexion

- the TGV Sud-Est between Paris and Lyon, opened in two stages in 1981 and 1983 and operated at 170 miles/h
- The TGV Atlantique which western branch to Le Mans opened in September 1989 and second branch to Tours in September 1990, at 190 miles/h
- the TGV Nord opened in May 1993 for a first half part; the whole line started operation September 1993, also at 190 miles/h.

The maximum speed on the TGV South-East line should shortly be raised to 190 miles/hr once vehicles and signalling systems have been adapted.

In spite of the relatively high speeds already possible on the existing conventional network, the time savings obtained with these new lines have reduced journey times, in some cases, by as much as half. Between Paris and Lyon average speed station-to-station is 135 miles/h, while on the TGV.A between Paris and Tours, it is 150 miles/h.

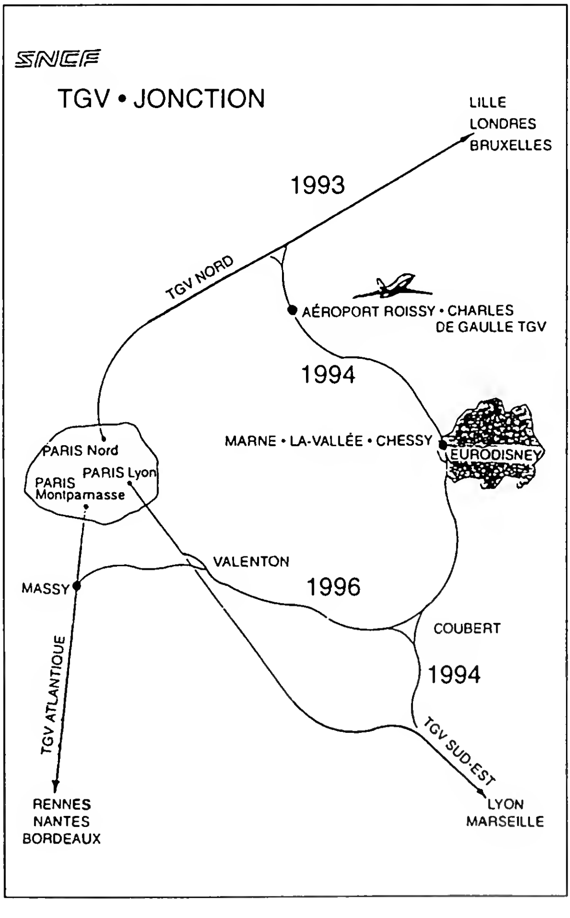


Figure 2

In October 1987, the French Government, in view of the technical, commercial, financial and economic success of the TGV.S.E., decided to have three further new lines built. These new lines are scheduled to come into service between 1992 and 1994.

The new lines will have a total length of 370 miles (see also Fig. 1)

- the TGV-Nord: Paris-Brussels/London via the Channel Tunnel (200 miles in France).
- the TGV-Rhone-Alpes: Lyon-Valence, the first extension of the TGV.S.E. towards the south
- the TGV Interconnexion (Fig. 2): a 65 mile line looping round Paris by the east to provide a high-speed link between the TGV Nord, Sud-est and Atlantique.

I.2. THE TGV NORD

The TGV Nord constitutes the French part of the North-European high-speed network connecting Paris-Brussels-Amsterdam-Cologne-Frankfurt and London, in conjunction with the Channel Tunnel. This project involves five countries (France, Great Britain, Belgium, the Netherlands and Germany).

The various sections are scheduled to be commissioned as follows:

- Paris-Lille-Calais: 1993 and London in 1994 by the Channel Tunnel
- Lille-Brussels: 1996
- the remaining sections: 1998.

In 1995, the traffic is expected to reach the following approximate levels:

- 17 million passengers between France and Great Britain
- 9 million passengers over domestic sections (Lille and the North of France)
- 6 million passengers between France and Belgium or beyond.

Time savings will be considerable—for example:

- PARIS-LONDON : 3 hours instead of 5 hours, 12 minutes
- PARIS-BRUSSELS : 1 hour 20 instead of 2 hours 25
- PARIS-LILLE : 1 hour instead of 2 hours.

Further savings in time may be possible in the longer term by building a new high-speed line between London and the Tunnel and a more direct line between Paris and the Tunnel. The combined effect of which would be to make Paris and London just 2 hours and 10 minutes apart.

I.3. The TGV Interconnexion

The new Interconnexion high-speed line is designed to sweep round to the east of Paris serving Roissy-Charles-de-Gaulle Airport and the Marne-La-Vallee/EuroDisneyland adventure park as it goes and ensuring continuity between the TGV Nord and Sud-Est. Connection to the TGV Atlantique will also be possible via, for a part of the way, an existing line section.

The result will be through high-speed services between large towns in the provinces and between these and major cities abroad, obviating the need to change trains and stations in Paris.

Therefore, all these high-speed lines form an integral part of the French railway network, interconnecting at several points with conventional lines. High-speed trainsets can therefore be worked through on to the existing network:

• to enter the heart of Paris conurbation and serve existing terminals

• to offer services to destinations beyond the end of the new lines.

The financial profitability to the TGV Nord is estimated at 12%; socio-economic profitability at 20%.

I.4. BUDGET

The budget for the whole operation is set at 3 billion US$ (not including tax) at 1989 prices (Fig. 3).

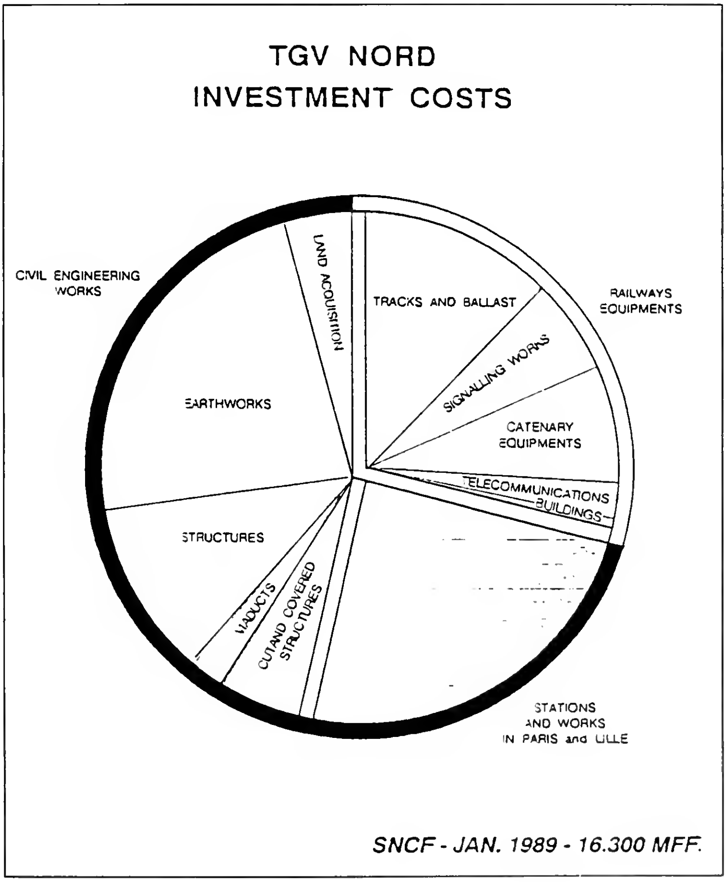


Figure 3

I.5. DETAILED DESIGN, LAND ACQUISITION

At the express request of the French Government, from 1988 onwards, the S.N.C.F. began to conduct implementation studies without waiting for the declaration of eminent domain status. These studies comprised:

- topographical, geographical and environmental surveys needed for the detailed studies
- meetings with local community and road management authorities to discuss details of situating and restoring the road network
- design of the line layout and longitudinal profile to a scale of 1:1000
- hydraulic surveys to establish civil engineering works required to restore the water network
- survey of the various networks affected by the project (i.e. water supply, drainage, gas, electricity, telephone, oil pipelines)
- discussions with professional agricultural organizations
- land registry inquiries designed to inform the public about the project's right of way, including a list of property involved and its owners. It is at this stage that individual and collective local interests are open to discussion.
- Over 10 square miles of land were expropriated from 4000 owners. Prior to this land registry surveys were held and lengthy negotiations took place with agricultural organizations and land tax authorities in order to establish the bases for compensation of land-owners and farmers.

Through these various agreements, it was possible to settle virtually all the cases of land expropriation without dispute; compulsory purchase was needed in less than 1% of all instances.

- Lastly, the detailed design studies define civil engineering structures and specific environmental protection measures needed. These were contained in the overall project file submitted for ministerial approval, and were the subject of the tenders launched for the individual items of work.

I.6. CONSTRUCTION WORK

All these studies, and also the land expropriation, were carried out in accordance with a work planning drawn up in 1987.

This planning schedule mad allowance for time needed for the different stages of the project:

- the Ministry's assessment and preparation for the public inquiry
- declaration of eminent domain status one year after the end of the inquiry
- expropriations, after state approval
- civil engineering work (construction of the track subgrade) during the two years after possession of the land
- permanent way works (track, signalling, electrifications, buildings) and testing, two years after completion of the civil engineering works.

The total length of time taken for implementation work after the declaration of eminent domain status is about four years (Fig. 4).

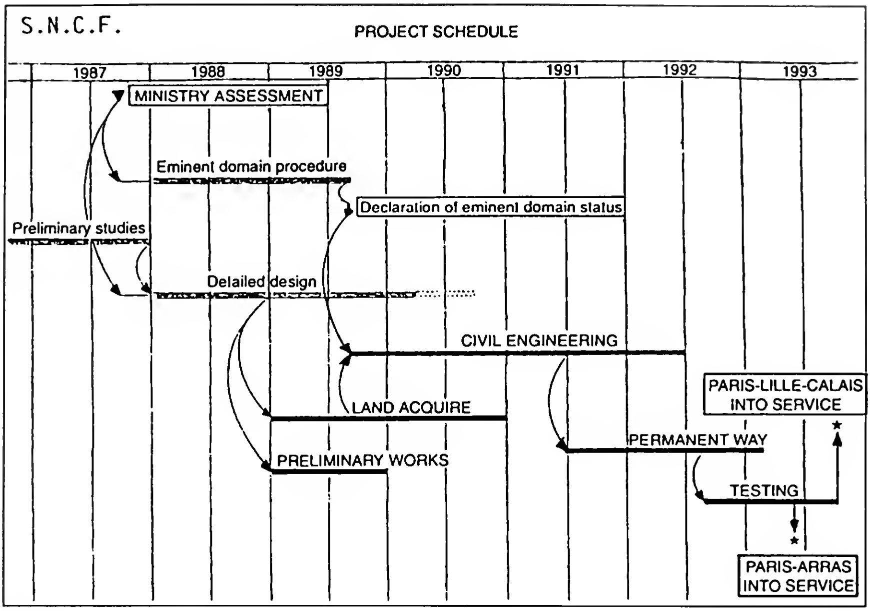


Figure 4

II. NEW TECHNOLOGY DEVELOPMENTS ON THE NORTH HIGH SPEED LINE:

II.1. COMPOSITE (METAL/CONCRETE) BRIDGES

Civil engineering works linking communication tracks must ensure that traffic movements enjoy the same degree of safety and comfort as that which existed before. This is all the more true when high-speed travel is involved as the dynamic effects underscore every occurrence likely to influence safety and, in dealing with high technology transport, it would be inconceivable to permit anything less than optimal comfort.

The S.N.C.F. already has a significant outlay of civil engineering structures on currently operated TGV lines, some of which have even been crossed at speeds exceeding 300 miles/h. However, these structures are all made of reinforced concrete, a prestressed concrete, or encased beams. As with advancements achieved in recent years in metallic construction, observations made with regard to this first family of structures have encouraged S.N C F. to make plans for several metallic or composite viaducts on high-speed lines in construction especially on the North New TGV lines now in operation, partly because they were not economically justifiable and also because of the fact that preliminary studies carried out to assess their behavior at high speeds had not made enough progress.

II.2. CAB SIGNALLING

Development of a more powerful cab signalling system for French National Railways' TGV Nord and the Channel Tunnel involved a switch from simple coded track circuits to a form of track-to-brain data transmission using 27-bit modulated messages.

When development work started on the Paris-South-East high speed line in the mid-1970s, French National Railways decided that cab signalling would be essential for trains travelling at 170 miles/h. S.N.C.F. developed the TVM 300 system, which was subsequently adapted for 200 miles/h services on the TGV Atlantique route.

Several fundamental principles were agreed when TVM 300 was developed, and these have been retained in the design of TVM 430:

- The line is divided into block sections, and signalling data is displayed in the cab in the form of speed indications transmitted block by block. Stopping distances cover several blocks. Signalling data is continuously transmitted by modulation of the carrier frequency modulations are available for sending 18 different speed-related data items to the train.
- Automatic train protection is incorporated into the cab signalling system, but will normally only come into action if the driver fails to respond correctly to signal indications. While the driver is complying with the cab indications, he has full control of the train (Fig. 5).
- Points control and speed indications are provided through interlockings around 13 miles apart and by intermediate location cabinets between each pair of interlockings. Both tracks are fully equipped for two-way working.

TVM 300 allows South-East services to run with a headway of 5 min., and Atlantic services every 4 min. These parameters were not considered good enough to cope with the anticipated volume of traffic on TGV Nord. S.N.C.F.'s technical departments were therefore asked at the end of 1987 to design a signalling system which would allow 3 min. headways at 200 miles/h.

TVM 430 standards

Apart from the 3 min. headway criterion, the development of TVM 430 was intended to meet a number of other objectives. In particular, provision was to be built in for a possible increase in maximum line speeds in the future. TGV Nord is being built for 220 miles/h, and the signalling is designed to accommodate further increases to 220 miles/h or more.

Another requirement was that TVM 430 must cater for various speed ranges, in order to maximise the use of constrained layouts on urban sections of the route. Equally, it must be compatible with existing sections of the route. Equally, it must be compatible with existing signalling, so trains equipped with TVM 430 will be able to run on the South-East and Atlantic lines.

To meet these objectives, three main alterations to the control structures were required. Firstly, the deceleration speed steps have been adapted. Secondly, the speed control curve has been replaced by the smoother, more elaborate curve, which matches more closely the braking curve normally followed by the driver. Lastly, a flashing signal has been added to the range of cab indications, in order to give advanced warning of the status of the signals in the next block when the train is approaching a speed reduction. This allows the drivers to start braking in advance of the main indication, and enables the block length to be reduced fom 1.25 miles to 1 mile on level track.

Data transmission

In order to expand the capacity of the cab signalling, it was necessary to increase substantially the number of data messages transmitted from ground to train. The principle of modulating the carrier frequency of the track circuits has been retained, but the number of different frequencies is now 27 compared to 18 used on the earlier lines. By combining the presence or absence of each of these frequencies in the modulation signal, 27-bit messages can be composed. 2 to the power of 21 items of useful information are available. This is a considerable improvement over the 18 basic messages transmitted by TVM 300.

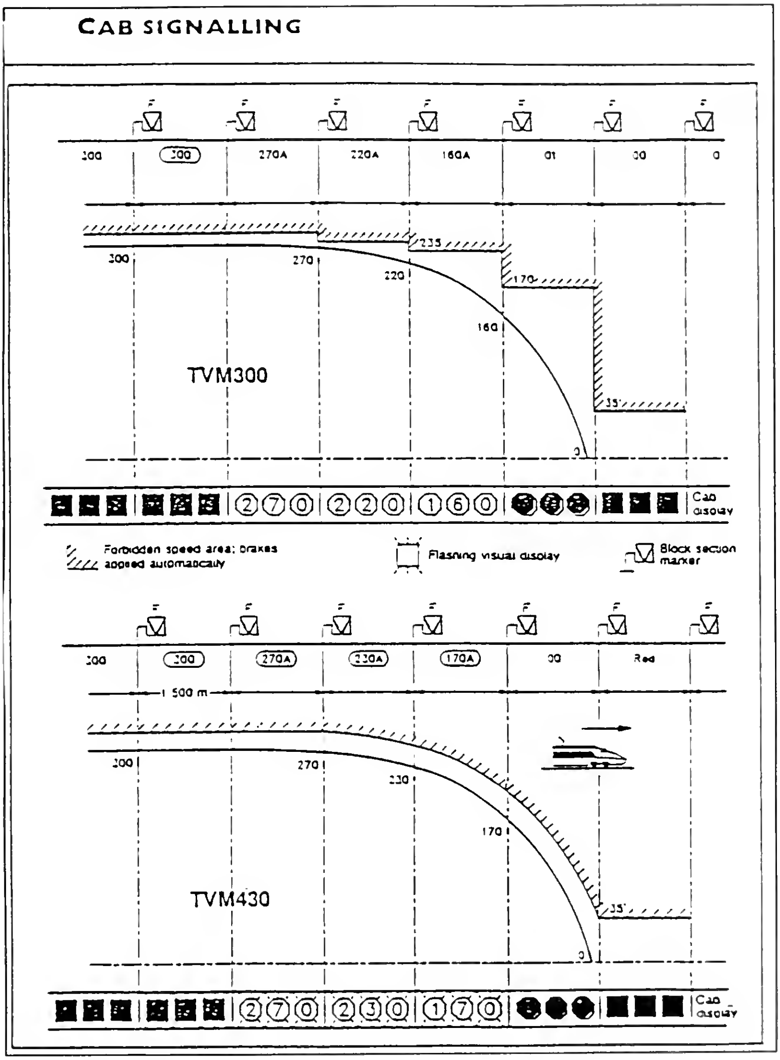


Figure 5

Ensuring safety

The first practical application of TVM 430 is the TGV-Nord.

Tests on the interlockings and ground-based installations started at the beginning of 1992, and the signalling system is ready for use on the line which is now opened for revenue service.

As the Channel Tunnel is also being equipped with TVM 430, cross-Channel services Eurostar will be able to run through from Paris to the British end of the Tunnel without change of signalling.

We have therefore also decided to adopt the system as standard for all further TGV lines to be built as part of the S.N.C.F. master plan proposals.

II.3. CONCRETE SLEEPER TURN-OUTS WITH WELDED MANGANESE FROGS

French railways developed in the 1980's a whole lot of new turn-outs laid on prestressed concrete sleepers.

These new turn-outs present a better dynamic behaviour under high speed trains.

After several years of tests on conventional lines and on the South-East High Speed Line, the first industrial production was laid on the North New Line and on the Interconnexion Line around Paris.

Furthermore, all the turn-outs built in France are now equipped with welded manganese steel forges and movable nose on the high speed lines.

The welding of solid cast manganese steel to the adjacent rails requires two problems to be solved:

- a problem of shape, with a design of frogs preventing welded zones to be overstressed
- a problem of metallurgy to ensure a good transition between the perlitic structure of rail steel and the austenite of the frog. In our process, a piece of stainless steel rail is flash-butt welded on each frog end to make a short insert—about 10 mn—after welding the adjacent rails.

III. HIGH SPEED LINES AND ENVIRONMENT

Last point now with some words about environmental considerations.

Respect for the environment is one of the train's major assets. France, like most other countries, is becoming more and more concerned with environmental issues. The train, more than any other form of transport, offers a positive and protective response.

The specific measures taken by S.N.C.F. in favour of environmental protection fall into 4 theme-based categories:

III.1 AGRICULTURE

After adoption of the definitive layout, restructuring measures designed to reduce the inconveniences of the new discontinuity brought about by the line and by reductions in the size of holdings affected by the TGV infrastructure, are studied and implemented in the usual way.

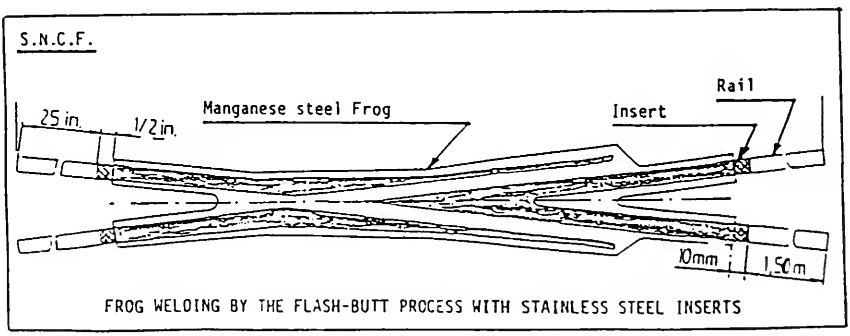


Figure 6

For the TGV Nord as a whole, restructuring work covered some 200 square-miles, the idea being to improve the productivity of farming units through concentration of plots of land around operating centres.

The problem posed by the drainage systems of farm holdings was duly studied, and measures are systematically taken to ensure their operational efficiency.

III.2. FLORA AND FAUNA

The TGV line layout is designed to safeguard forest-land as much as possible. It was finally adopted after sometimes difficult arbitrations between forests, arable land and residential areas.

The flora and fauna, the biotopy of which stood to be perturbed by the new line, were studies on a global basis as part of the preparation of the public-utility file and its accompanying environmental impact file.

The new line has a sufficiently-limited impact to avoid any risk of threatening the biotopies of large wild mammals. Alone the discontinuity effect created by the new line (which is completely fenced-in) can perturb the mobility of these animals. In the light of the study, it was decided to build 4 specific overpass structures on the North TGV Line and 2 underpasses.

The characteristics of these structures are specifically defined on the strength of scientific information available on the behaviour pattern of animals, and on the basis of experience gained n building such structures across motorways (Fig. 7).

III.3. THE NATIONAL HERITAGE

Unlike a conventional railway line with its very rigid linear profile in relation the lay of the land (slopes or gradients of some 8 mm/m), the construction of a TGV-type line enables the track to blend more harmoniously with its natural surrounds. The slopes of 25 mm/m (35 mm/m on the South-East Line), allow some valleys to be crossed at a very low height above the highest water levels.

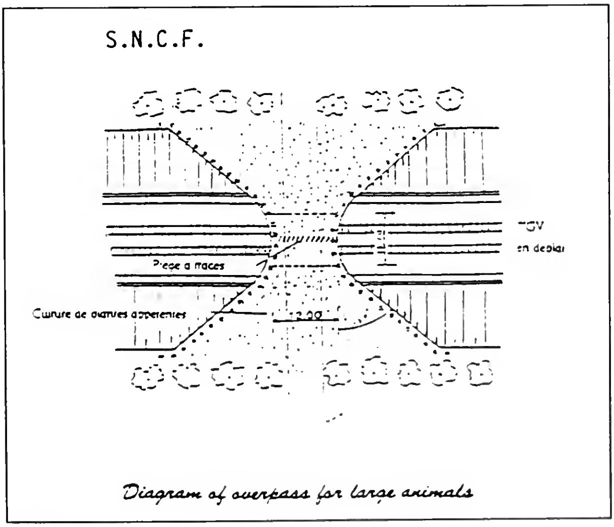


Figure 7

Architectural unity was sought in the design of the 364 civil engineering structures along the new line. The most important structures, calling for particular care as concerns their integration within the site, were designed with the help of noted architects.

Moreover, in conjunction with the Archeology Services of French Ministry of Culture, a stocktaking campaign based on aerial photography organized as part of the TGV project enabled sensitive zones to be identified before work actually commenced.

80 zones were cleared early enough so that the necessary evaluations and retrieval operations could be carried out, with the proviso of course that the archeologists themselves pledged not to delay the progress of work.

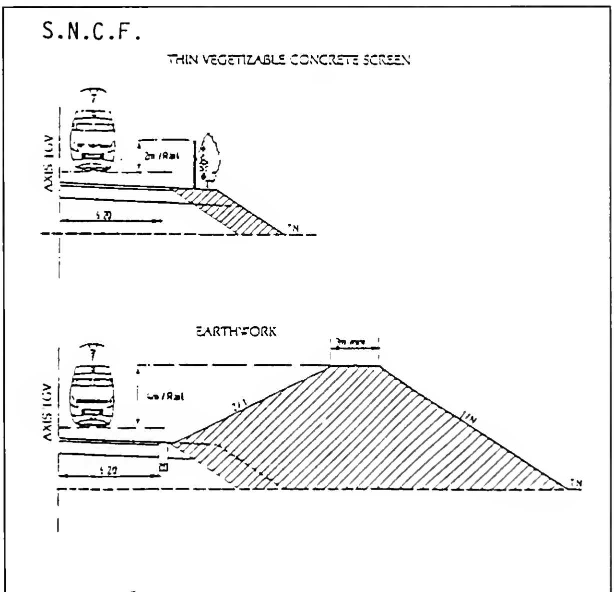
III.4. NOISE PROTECTION

Experience gained with the TGV Atlantique has enabled S.N.C.F. to develop proper systems for calculating predictable noise levels in relation to the different geographical configurations.

The efficiency of the different noise absorbing screens has also been fully tested in real-time conditions. This in particular has made it possible to verify that the creation of a screen—however modest in size (generally 2 or 3 m above rail level) is a most efficient means, if close to the track, of reducing the noise level.

In residential areas, the protections envisaged are of the earth-work type or formed of thin concrete screens (Fig. 8).

In order to determine the sites qualifying for such noise protection measures, a systematic study was carried out so as to evaluate the national noise level received by the front of lineside dwellings along the whole length of the line and located within a half mile strip centred on the platform.



Noise abatement measures adopted

Figure 8

This study has proved extremely useful for identifying the different at-source protection systems needed to observe the above thresholds.

Moreover, a few isolated properties will be protected by means of treatments applied to house fronts.

A total of:

- 26 miles noise absorbing earthworks
- 20 miles noise absorbing screens

are to be installed on the new line.

III.5. THE PRICE OF ENVIRONMENTAL CONCERN

In conclusion, we concentrate our efforts on four main areas:

- protection against noise pollution
- respect for green spaces and sensible landscapes
- the safekeeping of ecosystems
- respect of archeological heritage.

Spending related to environmental protection has risen steadily with each new project. It reached 7% of total costs (excepted railway equipments) on the TGV Sud-Est; it will represent 12% on the TGV Atlantique and 16% on the TGV Nord.

Norms cannot be established in this areas. The environment is beyond price but Railways have to satisfy three requirements. First, all legislation concerning environmental matters must be respected. Second, the company needs to take into account the requirements of regional populations affected by new lines. Finally, the profitability of the projects has to be safeguarded.

In this way, we have to develop new technology to improve the performances of our railways, to reduce investments and operating costs and to ensure security and reliability in operating the system.

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