TRANSPORTATION RESEARCH RECORD 744

Railroad Track and Facilities

TRANSPORTATION RESEARCH BOARD

COMMISSION ON SOCIOTECHNICAL SYSTEMS
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY OF SCIENCES
WASHINGTON, D.C. 1980

Transportation Research Record 744
Price $5.20
Edited for TRB by Mary McLaughlin

mode
3 rail transportation

subject areas
21 facilities design
40 maintenance
51 transportation safety

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Railroad track and facilities.

(Transportation research record; 744)
1. Railroads–Track–Congresses. I. Title. II. Series.
TE7.H5 no. 744 [TF240] 380.5s [625.1'4] 80-18849
ISBN 0-309-03052-8 ISSN 0361-1981

Sponsorship of the Papers in This Transportation Research Record

GROUP 2–DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
R. V. LeClerc, Washington State Department of Transportation, chairman

Railway Systems Section
Thomas B. Hutcheson, consultant, Jacksonville, Florida, chairman

Committee on Track Structure System Design
Paul S. Settle, Portec, Inc., chairman
Richard G. McGinnis, Bucknell University, secretary
Gilbert L. Butler, Louis T. Cerny, C. Page Fisher, Russell E. Frame, Amir N. Hanna, John B. Heagler, Jr., Wm P. Hofmann, James D. Jardine, Ben J. Johnson, Arnold D. Kerr, W.S. Lovelace, R. Michael McCafferty, Donald P. McConnell, James F. Pearce, Gerald P. Raymond, Allen L. Sams, Robert N. Schmidt, Alfred E. Shaw, John B. Smythe, Marshall R. Thompson, George H. Way, Jr.

GROUP 3–OPERATION AND MAINTENANCE OF TRANSPORTATION FACILITIES
Adolf D. May, University of California, Berkeley, chairman

Task Force on Railroad Operations
John C. Davis, Atchison, Topeka and Santa Fe Railway Company, chairman
Donald G. Capelle, P.J. Detmold, Steven R. Ditmeyer, Richard F. Dunlap, Harold H. Hall, James E. Martin, Carl D. Martland, Robert S. Ratner, Guerdon S. Sines, Otto F. Sonefeld, Charles E. Taylor, Dennis B. Wierdak

Task Force on Railway Maintenance
Richard F. Beck, Elgin, Joliet and Eastern Railway Company, chairman
Donald J. Bertel, James R. Blacklock, H.D. Campbell, O.L. Coy, Gordon H. Geiger, Milton E. Harr, Thomas B. Hutcheson, R. Michael McCafferty, William B. O'Sullivan, William R. Paxton, James F. Pearce, John F. Piper, Albert J. Reinschmidt, Paul S. Settle, Walter W. Simpson, Jr., Charles Slavis, George Van de Water, George H. Way, Jr.

Edward J. Ward and Thomas H. Myers, Jr., Transportation Research Board staff.

Sponsorship is indicated by a footnote at the end of each report. The organizational units and officers and members are as of December 31, 1979.

Contents

Authors of the Papers in This Record

Berger, Harold, Office of Nondestructive Evaluation, National Bureau of Standards, Physics Building, Room B312, Washington, DC 20234

Elliott, Carola, SRI International, 333 Ravenswood Avenue, Menlo Park, CA 94025

Fazio, A.E., Consolidated Rail Corporation, Six Penn Center Plaza, Philadelphia, PA 19104

Frederick, C.O., Research and Development Division, British Railways Board, London Road, Derby DE2 8UP, England

Hamilton, W.R., Research and Development, Portec, Inc., 300 Windsor Drive, Oak Brook, IL 60521

Hargrove, M.B., Association of American Railroads, 1920 L Street, N.W., Washington, DC 20036

Harris, William J., Jr., Research and Test Department, Association of American Railroads, 1920 L Street, N.W., Washington, DC 20036

Jones, E.G., Research and Development Division, British Railways Board, London Road, Derby DE2 8 UP, England

Marich, S., BHP Melborne Research Laboratories, 245 Wellington Road, Clayton, Victoria 3168, Australia

McGinnis, Richard G., Department of Civil Engineering, Bucknell University, Lewisburg, PA 17837

Mitchell, F.S., Dynelectron Corporation at the Transportation Test Center, Pueblo, CO 81001; formerly with Association of American Railroads

Moody, Howard G., Federal Railroad Administration, U.S. Department of Transportation, 400 7th Street, S.W., Washington, DC 20590

Prybella, Robert, Sverdrup and Parcel Associates, Inc., 1625 I Street, N.W., Washington, DC 20006; formerly with Consolidated Rail Corporation

Rimnac, C.M., Lehigh University, Bethlehem, PA 18015

Sakasita, Masami, SRI International, 333 Ravenswood Avenue, Menlo Park, CA 94025

Schumacher, Robert, Mass Transit Planning, New York City Department of Transportation, 40 Worth Street, New York, NY 10013

Steele, R. K., Transportation Test Center, Federal Railroad Administration, Pueblo, CO 81001

Stock, William A., SRI International, 333 Ravenswood Avenue, Menlo Park, CA 94025

Stone, D.H., Association of American Railroads, 3140 South Federal Street, Chicago, IL 60616

Wong, Peter J., SRI International, 333 Ravenswood Avenue, Menlo Park, CA 94025

Young, R.E., H&H Aerospace, 51 Brattle Street, Suite 26, Cambridge, MA 02138

Zarembski, Allan M., Association of American Railroads Technical Center, 3140 South Federal Street, Chicago, IL 60616

Rail Research: Meeting the Challenge of Modern Traffic Loading

Allan M. Zarembski

The current trend in the railroad industry toward heavier cars and increased wheel loads and the subsequent effect of this trend on rail are discussed. Because of the increased loadings, the replacement criterion for rail in main-line tangent track has changed from rail wear to initiation of rail fatigue defects. An analysis of field data shows that this initiation of fatigue defects is a function of both wheel load and rail size. Rail research in North America and its thrust toward improving and extending rail service life are also discussed.

In light of the current trend in railroading toward heavier cars and trains, the railroad track structure is being called on to perform under an increasingly severe loading environment. As a result, the very nature of rail failure has changed. This change in the modes of rail failure has resulted in changes in criteria for rail replacement and consequently in changes in inspection and maintenance practices.

Track rail once lasted until it literally wore out. Under today's severe loads, however, fatigue-initiated cracks in the railhead can result in premature fracture of the rail. Furthermore, it is often not possible to see the fatigue crack, even at its critical point. Ultrasonic or magnetic inspection techniques must be used to detect these hidden defects so that they can be removed.

Rail-end batter, the traditional replacement criterion for tangent track, has been significantly reduced by the increasing use of continuously welded rail. In its place, however, fatigue-induced defects, either in the rail or at a weld, have emerged as the dominant rail-replacement criterion for tangent track. On curves, severe gage face wear, plastic flow of the railhead, and even crushed rail are all major problems that combine with initiation of fatigue defects to shorten the service life of rail.

To better understand the problem, one must only consider that a stationary 91-Mg (100-ton) car with a static wheel load of 146 kN [33 000 lbf (33 kips)] transmits a contact stress of 1200 MPa (175 000 lbf/in^2) to the head of the rail. The yield strength of the rail steel is only 520 MPa (75 000 lbf/in^2). The result can be seen in Figure 1: rail wear, fatigue defects, or both.

Thus, in recent years, rail research has been directed toward the problem of defining, quantifying, and ultimately extending rail service life. It is the purpose of this paper to briefly define and quantify some of these modes of rail failure and to discuss the current and future directions of rail research in North America.

FATIGUE DEFECTS

The modes of rail failure, particularly the types of rail defects that can occur and that result in rail failure, are amply described in the literature (1-3). In recent years, examination of track test sites—such as the six sites maintained by the American Iron and Steel Institute (AISI), the Association of American Railroads (AAR), and the American Railway Engineering Association (AREA) as part of the Joint Cooperative Rail Research Program on the Union Pacific and the Atchison, Topeka, and Santa Fe Railroads—has shown the growing predominance of the detail fracture-shell type of defect under mixed freight loading (4). This trend is clearly indicated in Figure 2, which shows that where there is a predominance of mixed freight traffic with some unit trains, defects of the detail fracture type tend to dominate the failure-inducing defects that result in rail replacement and rail maintenance.

Further examination of the occurrence of detail fracture defects with gross loading on the track indicates that there is a point at which the rate of defect occurrence increases dramatically (see Figure 3). This means that, after an initial period in service, there is a significant increase in the rate at which defects in rail occur. Recent probability analyses of defect data indicate that this increase occurs in the range of 182 million-636 million gross Mg (200 million-700 million gross tons) (5).

Evaluation of transverse defect data from the Facility for Accelerated Service Testing (FAST) at Pueblo, Colorado, where a unit-train type of operation is simulated, also shows this behavior (Figure 4). It should be noted, however, that the point of transition for the FAST data occurs at a significantly lower load level than for the mixed traffic cases shown in Figures 2 and 3 for the same size rail. This observation—that increased loadings, such as those produced by heavy 91-Mg (100-ton) cars in unit train service, results in reduction of rail service life—is supported by recent analyses of the fatigue life of rail (6, 7). These analyses, which use loads and stress values for different types of traffic together with data on the fatigue properties of rail material, indicate that as the severity of loading increases—i.e., as the size of the freight car and the corresponding axle loads increase—the fatigue life of rail in service decreases (see Figure 5). This effect agrees with general observations made in the field by track engineers (8) and clearly illustrates that the use of heavier cars results in a direct increase in maintenance costs. Thus, whatever benefits accrue from the use of freight cars with larger loading capacities, they must be balanced against these increased maintenance costs in order for a true cost/benefit comparison to be made (9, 10).

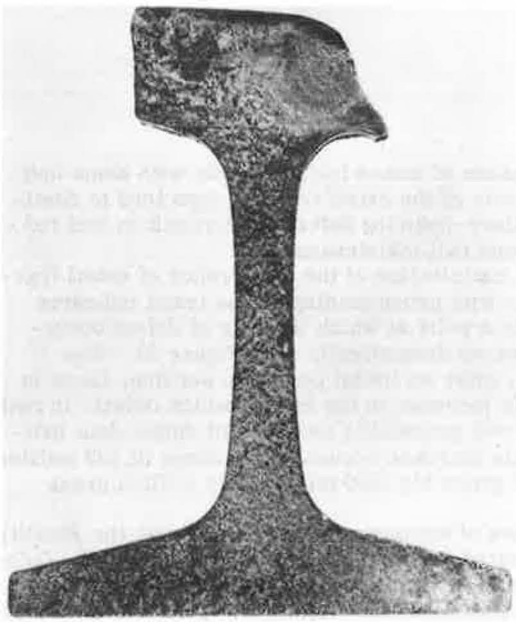
The analyses shown in Figure 5 also indicate that a definite benefit can be gained in rail service life by increasing the weight, and correspondingly the size, of the rail section in the track. This behavior is supported by field data such as those illustrated in Figure 3, which shows the number of defects versus car weight for two sections of track on the Atchison, Topeka, and Santa Fe Railway under similar traffic conditions but with different rail-section sizes. As Figure 5 further shows, this benefit of increased rail size occurs under differing traffic mixes and loading conditions as well.

These investigations, together with other ongoing work in the areas of fatigue failure, fatigue crack propagation, fracture mechanics, and rail stresses (11), represent the current state of the art in the study of fatigue defect behavior in rail steels.

RAIL WEAR

Rail wear remains the dominant criterion for rail replacement on curved track in North America. It also remains one of the most important causes of rail replacement. Thus, the ability to predict rail-wear life and to decrease the rate of rail wear has been of great

Figure 1. Rail exhibiting detail fracture combined with extreme curve wear.



concern to the track engineer.

In 1969, AREA developed an equation for the calculation of rail life based on railhead wear (12). This equation, which was empirically derived from field measurements, provided the following relation (since the equation was formulated in U.S. customary units of measurement, no SI equivalents are given):

$$T = KWD^{0.565} \quad (1)$$

where

T = life of rail in main-line track (million gross tons),
K = constant reflecting level of track maintenance and type and condition of track structure (average = 0.545),
W = weight of rail (lb/yd), and
D = annual tonnage density (million gross tons).

More recently, the Canadian Institute of Guided Ground Transport (CIGGT) has developed a rail-wear model that uses a combined empirical and analytic approach (13). This model has a capability for predicting rail wear that enables the user to define track and traffic conditions and obtain an analytic prediction of wear life. Such a prediction is shown in Figure 6, which also shows the effects of axle loads, rail heat treatment, and lubrication on rail wear.

Investigation into the mechanism of rail wear represents another approach taken in the understanding of wear and in the development of techniques to improve wear life. The recent work of several authors (14-16) represents the state of the art in the study of rail-wear behavior.

Additional research in the area of improving rail metallurgy, particularly that oriented toward improving characteristics of rail wear, is being pursued extensively both in North America and abroad (11). The results of this research, in particular the results on the different types of metallurgy and heat treatment, are being evaluated under service conditions and at FAST (17). Preliminary results from the accelerated service testing at FAST are shown in Figure 7. In the figure, each point on the curve represents the mean value of railhead area loss for a random mix of rail-cant and shoulder-width test sections. These preliminary results show that improved rail-wear characteristics can be obtained from improvements in rail steel.

WEAR VERSUS FATIGUE

As noted earlier, because of the current tendency toward heavier rail cars and increased wheel loads, the nature of rail failure in general, and the maintenance criterion for rail replacement in particular, are undergoing significant changes.

The emergence of the problem of the initiation and growth of fatigue defects in rail has resulted in major changes in rail inspection techniques and rail replacement practices. Because of the serious safety consequences of rail defects, rail must often be removed from track long before it has worn out, and this represents a serious economic consequence (9, 10).

A recent comparison of the wear life of 136 RE standard carbon rail with the fatigue life of the same rail for different wheel loads is shown in Figure 8 for tangent track. The nominal wheel load represents the largest static wheel load imposed on a section of track that experiences a defined mix of traffic. Actual dynamic loads that corresponded to these static values were then used to determine the respective lives. The fatigue line on this curve was obtained by using the slope of the S-N curve for rail steel and the calculated fatigue life of 136 RE rail under the defined loading conditions (7). The wear line was obtained by using the AREA wear formula (Equation 1).

Figure 8 shows that, as the static wheel load (i.e., the weight of a four-axle freight car) increases, the failure mode of the rail shifts from wear to fatigue. For traffic with nominal static wheel loads greater than 124 kN (28 000 lbf)—i.e., greater than 63-Mg (70-ton) traffic—fatigue failure emerges as the criterion for rail replacement. This appears to be in agreement with field experience (7).

Further examination of Figure 8 shows that the curve for fatigue life versus wheel load has a very steep slope. This suggests that there are significant rail-life penalties associated with additional increases in wheel loads. Figure 5 confirms this observation. Thus, any further increase in wheel loads should only be made after there is adequate understanding of both the safety and economic consequences (8, 9).

CURRENT RAIL RESEARCH

Rail research is currently directed toward understanding the mechanisms of rail failure and developing techniques to extend and improve rail service life. Recently, several conferences and presentations have examined the various aspects of rail research, both in North America and abroad (11, 18, 19). Therefore, only a brief overview of current, ongoing rail research in North America is presented here.

Current investigations into the failure mechanisms of fatigue and fracture in rail steel include empirical evaluations of test site data, such as those from the joint AISI-AAR-AREA test sites on the Union Pacific and the Atchison, Topeka, and Santa Fe Railroads (4, 11, 20, 21); FAST (17); and others (22). Also included are ongoing laboratory investigations into (a) cyclic fatigue behavior (Northwestern University), (b) fatigue and fracture of rail steel [Carnegie-Mellon University (11) and U.S. Steel (23)], and (c) residual stresses in steel and analyses of service-developed defects [U.S. Steel (21)]. Finally, analytic investigations, such as AAR analyses of fatigue-defect initiation (6, 7), and analyses of stresses around verse fissure flaws [Battelle-Columbus Laboratories (11)] are also ongoing.

In current research in rail wear and rail corrugation, ongoing studies of rail wear include basic evaluation of rail-wear mechanisms (a) on tangent track [Illinois Institute of Technology (16)], (b) on curved track [Colorado School of Mines (15)], and (c) in general service (14). Rail-wear modeling work is currently being pursued at the Canadian Institute of Guided Ground Transport (13).

Figure 2. Distribution of rail defects at Union Pacific test site.

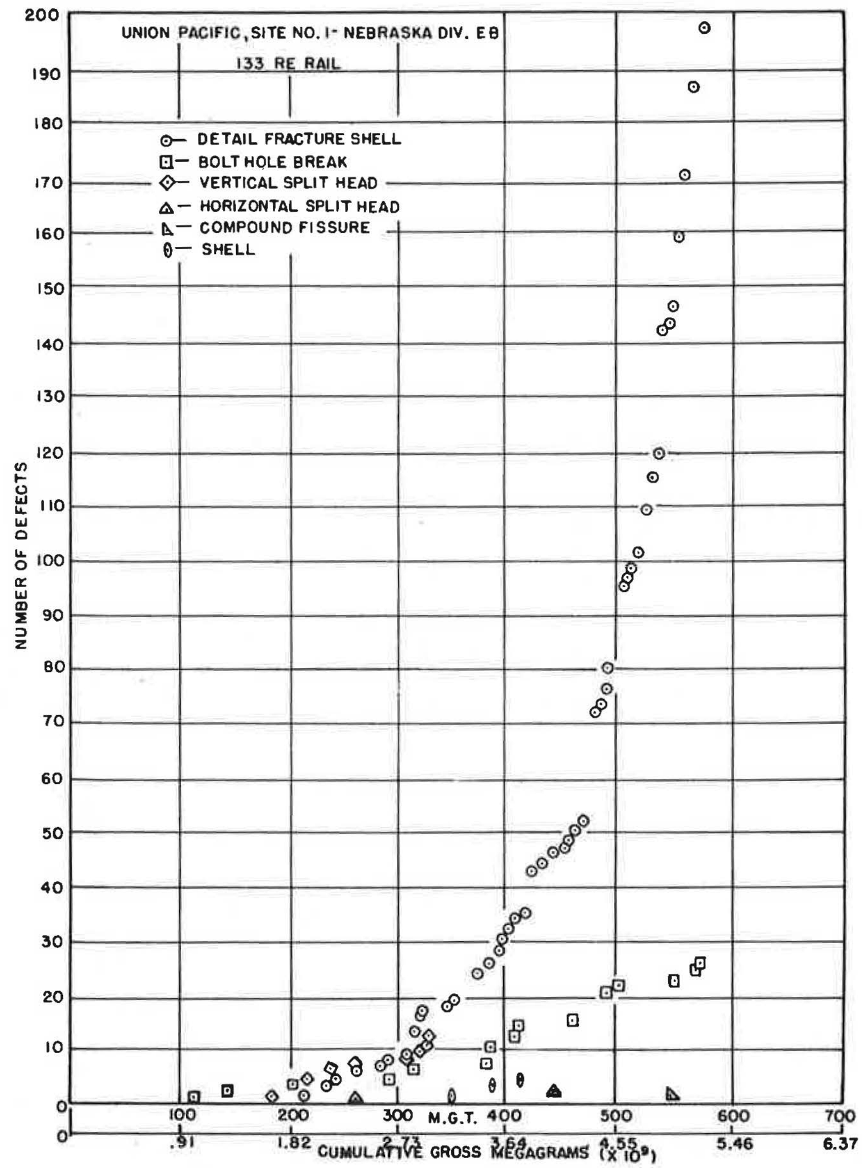


Figure 3. Cumulative rail defects versus cumulative traffic loading at Atchison, Topeka, and Santa Fe Railway test sites.

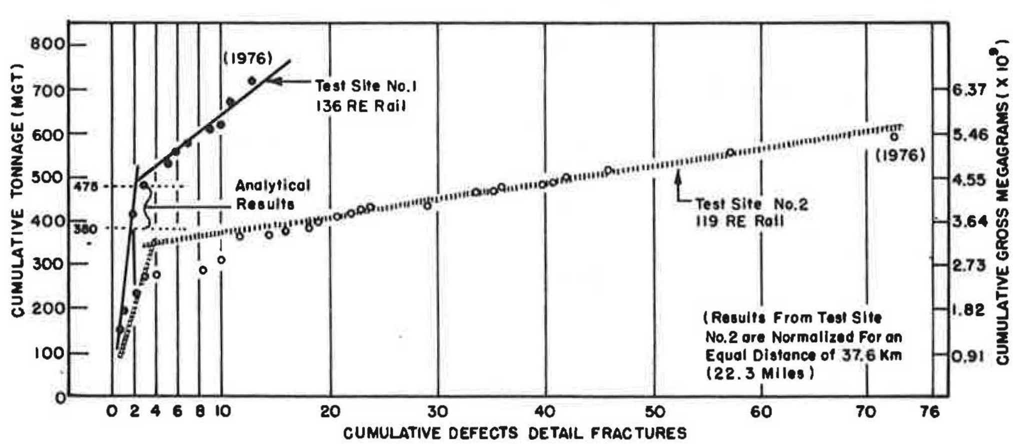


Figure 4. Cumulative rail defects at FAST.

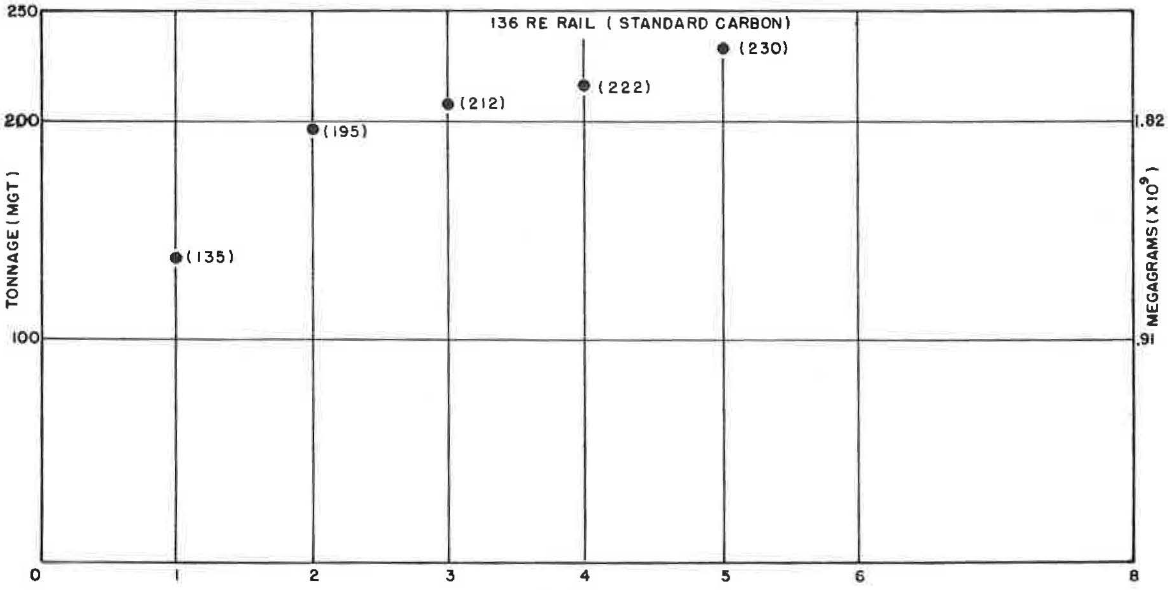


Figure 5. Effect of increasing traffic loading.

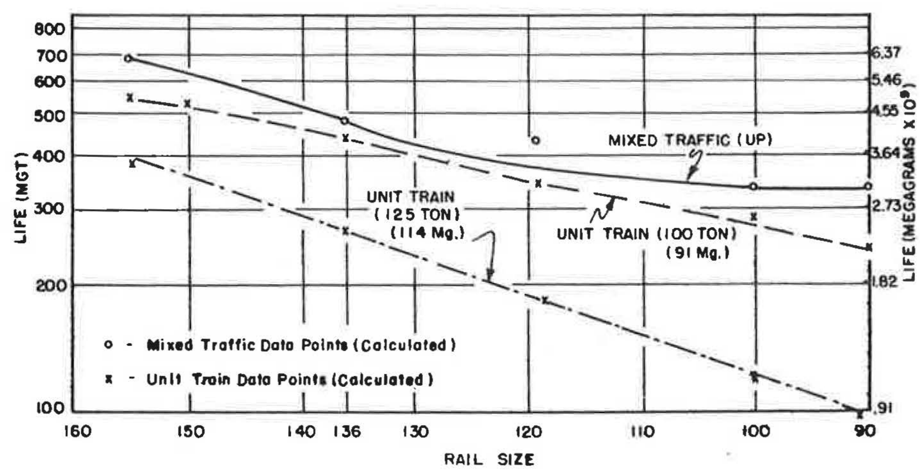
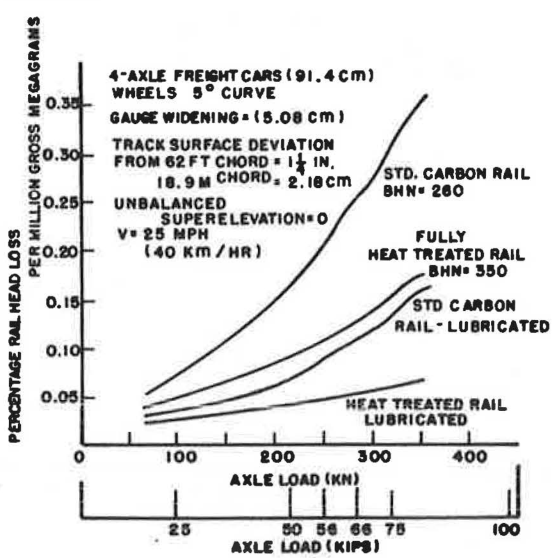


Figure 6. Prediction of rail wear by RAILWEAR 2 program of CIGGT.



Work on rail corrugation has been done by Kalousek (24) as part of the Track-Train Dynamics program. Finally, an empirical investigation of wear is ongoing at FAST, where the wear characteristics of different rail metallurgies are being studied (17). Also ongoing at FAST are investigations into the occurrence of rail corrugations.

Other rail research programs include investigation into and development of improved rail metallurgies (11, 19) and development of improved techniques of rail-flaw inspection (25, 26). In the latter area, both AAR and the Federal Railroad Administration are particularly emphasizing the extension of existing ultrasonic and magnetic inspection techniques to increase depth of penetration and speed of inspection. Work in the area of inspection of field and plant welds is also being pursued. This effort, together with studies to improve the Thermit welding process (at Arizona State University) and to develop homopolar welding techniques (at the University of Texas), is aimed at expanding and improving the state of the art in rail welding technology.

Figure 7. FAST comparison of rail wear for different rail metallurgies.

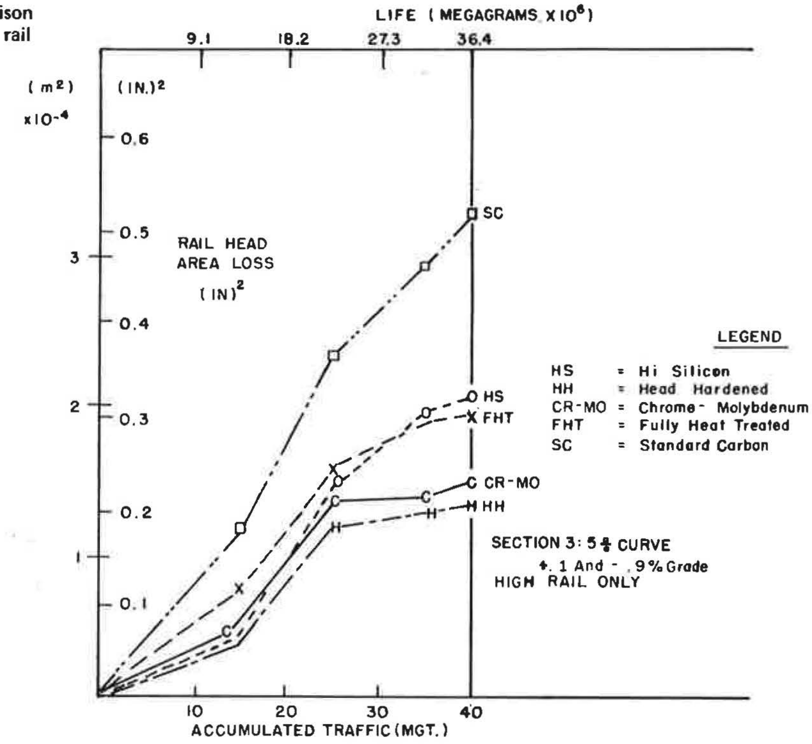
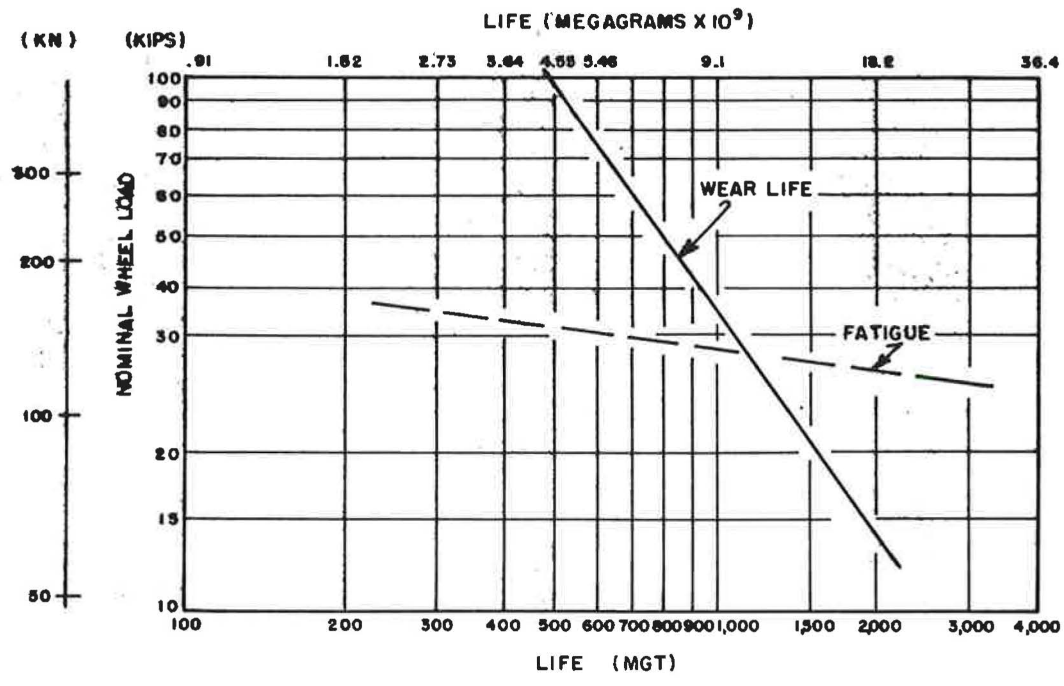


Figure 8. Rail-life fatigue versus wear for 136 RE standard carbon rail.



SUMMARY

The railroad track structure, and particularly the rail itself, are being called on to perform under increasingly severe loading conditions. Consequently, new modes of rail failure are demanding that the railroads improve, and in many cases change, their basic maintenance practices. As these new failure modes become more prevalent, railroads are finding that a more complete understanding of rail behavior under load is necessary to improve rail performance. This is the objective of current rail research. This paper is intended to serve as a brief introduction to the problem of decreasing rail service life under increasing traffic loading and to the efforts of rail research in searching for solutions to that problem.

REFERENCES

1. Rail Defect Manual. Sperry Rail Service, Danbury, CT, 1964.
2. W. W. Hay. Railroad Engineering. Wiley, New York, Vol. 1, 1953.
3. Railway Track Materials. American Iron and Steel Institute, New York, Oct. 1975.

4. P. S. Besuner and others. Rail Analysis: Volume 3—Statistical Analysis of Rail Defect Data. Assn. of American Railroads, Track-Train Dynamics Rept. R-302, Chicago, June 1978.
5. P. S. Besuner and others. Probability Analysis of Rail Defect Data. Presented at Heavy Haul Railways Conference, Perth, Australia, Sept. 1978.
6. A. M. Zarembski. Effect of Rail Section and Traffic on Rail Fatigue Life. Presented at AREA Tech. Conference, Chicago, March 1979.
7. A. M. Zarembski and R. A. Abbott. Fatigue Analysis of Rail Subject to Traffic and Temperature Loading. Presented at Heavy Haul Railways Conference, Perth, Australia, Sept. 1978.
8. J. R. Sunnygard. Effect of Heavy Cars on Rail. Presented at AREA Tech. Conference, Chicago, March 1977.
9. G. H. Way. Economics of Freight Car Size. Presented at AREA Tech. Conference, Chicago, March 1979.
10. R. E. Ahlf. Heavy Four-Axle Cars and Their Maintenance-of-Way Cost. AREA Bull., Vol. 76, June-July 1975.
11. D. H. Stone and G. G. Knupp. Rail Steels: Developments, Processing, and Use. ASTM, Philadelphia, Special Tech. Publ. 644, May 1978.
12. Determination of Maintenance-of-Way Expense Variation with Various Traffic Volumes. AREA Bull., Vol. 70, 1969, p. 81.
13. M. D. Roney and others. Road Maintenance Cost Model: Phase 1—Rail Wear Modeling. Canadian Institute of Guided Ground Transport, Kingston, Ontario, Rept. 78-2, 1978.
14. J. Kalousek and A. E. Bethune. Rail Wear Under Heavy Traffic Conditions. In Rail Steels: Developments, Processing, and Use, ASTM, Philadelphia, Special Tech. Publ. 644, May 1978.
15. W. E. Jamison. Final Summary Report on Mechanical Wear of Railroad Components. Assn. of American Railroads, Chicago, Jan. 1979.
16. S. Kumar and R. Margasahagan. Quantitative Wear Analysis of Simulated Steel Wheel and Rail. Assn. of American Railroads, Chicago, March 1978.
17. Facility for Accelerated Service Testing Progress Report 2. Assn. of American Railroads, Chicago, Rept. R-288, Sept. 1977.
18. Heavy Haul Railways Conference. Australia Institution of Engineers and Australian Institute of Mining and Metallurgy, Perth, Sept. 1979.
19. S. Marich. Research on Rail Metallurgy. AREA Bull., Vol. 78, June-July 1977.
20. D. P. Johnson and P. S. Besuner. Rail Analysis: Volume 2—Engineering Cost-Risk Analysis of Defective Rail. Assn. of American Railroads, Chicago, Track-Train Dynamics Rept. R-265, June 1978.
21. D. E. Sonon and others. Rail Analysis: Volume 4—Metallurgical Examination of Rails with Service-Developed Defects. Assn. of American Railroads, Chicago, Track-Train Dynamics Rept. R-300, March 1978.
22. P. S. Besuner. Rail Analysis: Volume 1—Fracture Mechanics. Assn. of American Railroads, Chicago, Track-Train Dynamics Rept. R-225, June 1978.
23. J. M. Barsom and E. J. Imhof. Rail Analysis: Volume 5—Fatigue and Fracture Behavior of Carbon-Steel Rails. Assn. of American Railroads, Chicago, Track-Train Dynamics Rept. R-301, March 1978.
24. J. Kalousek. Track-Train Dynamics Report 4: Rail Corrugations. Canadian Pacific, Ltd., Montreal, Res. Rept. S-488-75, Feb. 1975.
25. G. Falkenbach. Improvements in Magnetic Techniques for Rail Inspection. Transportation Systems Center, U.S. Department of Transportation, Cambridge, MA, Rept. DOT-TSC-1244-1, Dec. 1977.
26. A. K. Gurvich. Flaw Detection in Rails. Federal Railroad Administration, U.S. Department of Transportation, Rept. FRA-ORD-78-10, Dec. 1978.

Publication of this paper sponsored by Committee on Track Structure System Design.

Evaluation of Rail Behavior at the Facility for Accelerated Service Testing

M. B. Hargrove, F. S. Mitchell, R. K. Steele, and R. E. Young

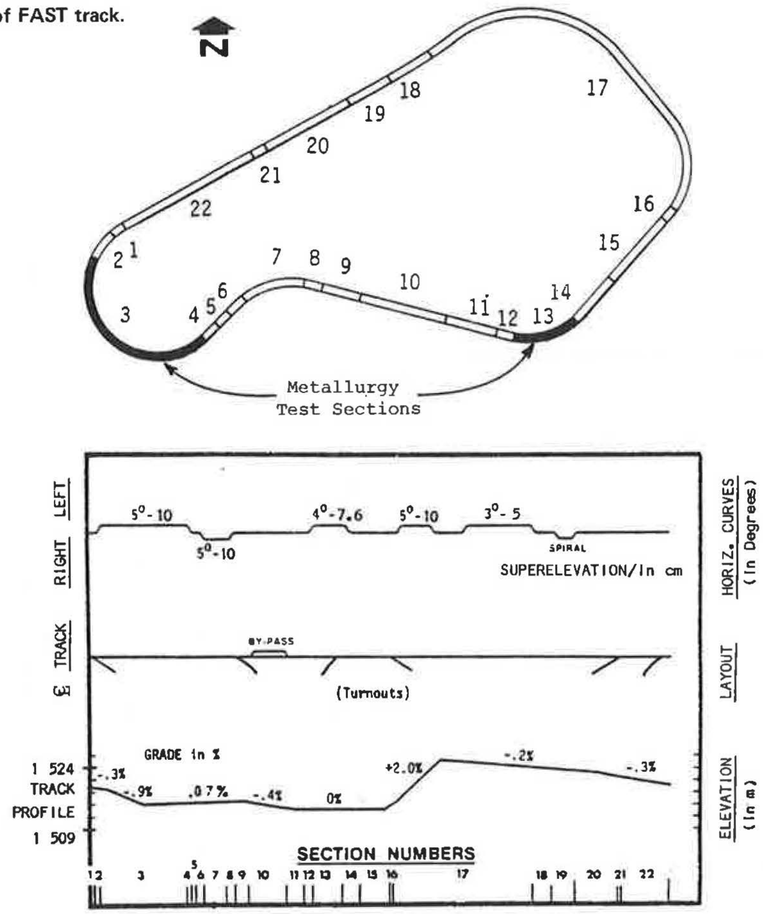
Results of two experiments conducted at the Facility for Accelerated Service Testing to investigate the wear and defect behavior of various rail metallurgies under unit train operations are presented. Five types of rail were used: standard carbon, high-silicon, head-hardened, chrome molybdenum, and fully heat-treated. The load demarcation between the two experiments was at a traffic loading of 122 million gross Mg (135 million gross tons). In the first experiment, a condition of underlubrication existed up to 36 million-41 million gross Mg (40 million-45 million gross tons), after which point lubrication could be described as generous, a condition maintained throughout the second experiment. Railhead profile measurements taken in both experiments revealed that head-hardened and chrome molybdenum rail exhibited the best resistance to high-rail curve wear. In the first experiment, there was a strong lubrication-metallurgy interaction that caused the premium metallurgies to benefit less than standard rail from generous lubrication. In the underlubricated condition, the 1:14 tie-plate cant produced about 20 percent more gage-face and head-area loss than the other cants. The cant effect was considerably reduced by generous lubrication. Position-in-curve effects were dependent on the level of lubrication. When generous lubrication permitted the accumulation of greater loads on the rails, fatigue failure became the dominant failure mode in both railhead and weldments. Standard rail exhibited the greatest number of failures from railhead fatigue.

The rail metallurgy experiment at the Federal Railroad Administration's Facility for Accelerated Service Testing (FAST) has as its primary intent the development of information on rail wear in a controlled environment. However, useful information on rail and weld failure behavior has also been forthcoming.

The FAST track (see Figure 1) is a specially constructed 7.7-km (4.8-mile) loop divided into 22 sections in which specified combinations of track components and structures are installed for testing. It contains 3.5 km (2.2 miles) of tangent, 0.64 km (0.4 mile) of 3° curve, 0.48 km (0.3 mile) of 4° curve, and 1.8 km (1.1 miles) of 5° curve; the remaining 1.29 km (0.8 mile) is in transitional spirals.

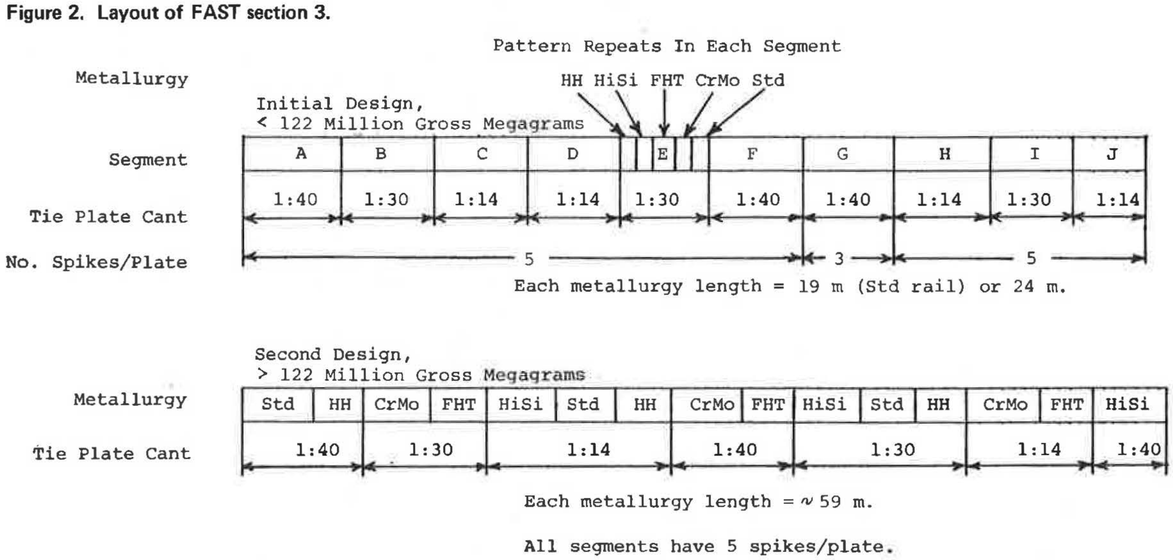
The FAST consist is made up of four-axle locomotives

Figure 1. Diagram and profile of FAST track.



1 km = 0.62137 mi
1 m = 3.2808 ft

Figure 2. Layout of FAST section 3.

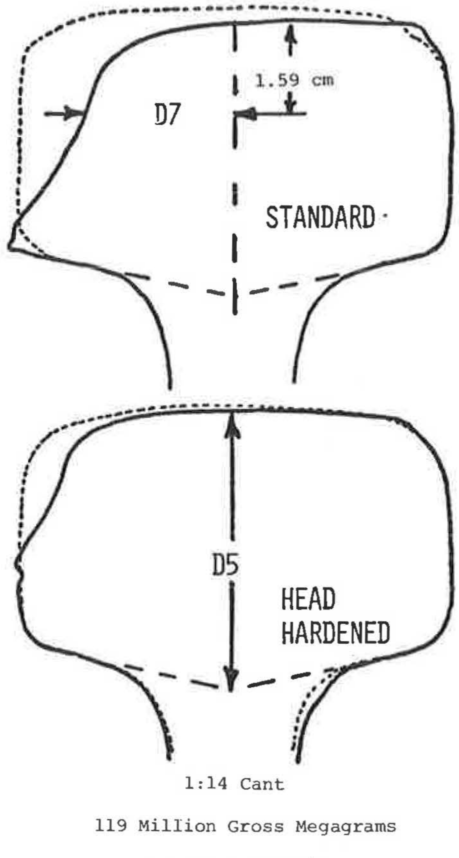


1 m = 3.281 ft

Table 1. Average chemical analyses of rail in first FAST metallurgy experiment.

		w/o						
Section	Metallurgy	Carbon	Manganese	Phosphorus	Sulfur	Silicon	Chromium	Molybdenum
3	HH	0.79	0.84	0.009	0.018	0.16	-	-
	HiSi	0.76	0.86	0.028	0.027	0.63	-	-
	FHT	0.69	0.81	0.018	0.032	0.18	-	-
	CrMo	0.80	0.82	0.026	0.025	0.25	0.78	0.20
	Standard	0.78	0.86	0.027	0.025	0.15	-	-
13	HH	0.77	0.88	0.015	0.025	0.18	-	-
	HiSi	0.77	0.88	0.029	0.024	0.68	-	-
	FHT	0.77	0.81	0.020	0.041	0.15	-	-
	Standard	0.73	0.86	0.024	0.020	0.17	-	-

Figure 3. Measurements and extremes of railhead profile.



that normally haul a 75-car, 8616-Mg (9500-ton) train. The majority of the cars are 91-Mg (100-ton) hopper or gondola cars; the remainder are 91-Mg-capacity tank cars and laden trailer on flat cars. The average train speed is 65.9 km/h (41 miles/h), and the maximum speed is 72.4 km/h (45 miles/h).

Tests are run five days a week. Each test run begins in the afternoon, continues all night, and ends the next morning. On each run, the consist makes approximately 120 laps of the track, imposing a traffic loading of about 0.907 million gross Mg (1 million gross tons) on the track and putting about 966 km (600 miles) on the cars (throughout this paper, loads are in gross megagrams and gross tonnage).

The rail metallurgy tests have been conducted on track sections 3 and 13. Section 3 is a 1097-m (3600-ft) long 5° curve. The north half is on a 0.9 percent grade, and the south half is virtually level. Section 13 is a 381-m (1250-ft) long 4° curve that is level throughout its length (Figure 1).

As of June 1979, two rail metallurgy experiments had been completed. The first extended until 122 million Mg (135 million tons) was imposed on the track, and the second extended for an additional 263 million Mg (290 million tons) until a load of 385 million Mg (425 million tons) was imposed. At the beginning of the first experiment, rail lubrication was provided from track lubricators in sections 5 and 17 only. However, between 27 and 45 million Mg (30 and 50 million tons), two additional lubricators were installed in sections 2 and 18, and the section 17 lubricator was moved to section 14. Thus, two regimes of behavior, one of underlubrication and the other of generous lubrication, existed in the first experiment, the transition occurring at 36-41 million Mg (40-45 million tons). In the second experiment, only the generous lubrication was used.

The physical layouts of the first and second experiments in section 3 are shown in Figure 2. Section 3 was laid with American Railway Engineering Association (AREA) standard carbon, high-silicon (HiSi), chrome molybdenum (CrMo), fully heat-treated (FHT), and head-hardened (HH) rail, all in rail sections of 65.5, 67.5, or 69.5 kg/m (132, 136, or 140 lb/yd). In both experiments, the same five metallurgies and three tie-plate cants were the primary test parameters. However, the change in physical layout was necessitated by an unexpectedly large number of weld failures in the first experiment and by a concern that the short lengths of each metallurgy segment in the first experiment—19-24 m (62-78 ft)—would produce nonrepresentative wear results. Possible position-in-curve effects were compensated for by providing at least three replications of the metallurgies around the curves.

In each experiment, section 13 was configured almost identically to section 3 except for a reduced number of metallurgies and the use of 1:40 tie-plate cant throughout. In the first experiment, four metallurgies were tested: standard, HiSi, FHT, and HH, each replicated four times. All rail in section 13 was 57.1 kg/m (115 lb/yd).

The average ladle chemistries of the rails tested in the first experiment are given in Table 1. With the exception of the FHT rail tested in section 3, which had a carbon content slightly less than 0.70 percent by weight (w/o), and the standard rail tested in section 13, which had a carbon content slightly less than 0.75 w/o, all metallurgies had average carbon contents in the range of 0.76-0.80 w/o. The average manganese levels ranged from 0.81 to 0.88 w/o.

Wear measurements were taken by using profilometers that produce 1:1 tracings of the railhead, beginning at the fishing surface under one side of the head and tracing around to the other. Correction procedures were applied to compensate (sometimes only partly) for instrument and operator variability. In addition, a change in the correction procedure was introduced at

72.5 million Mg (80 million tons), and this change was believed to be responsible for the transients in wear data observed at that load. Measurements were taken at intervals of approximately 18 million-23 million Mg (20 million-25 million tons), at a minimum of two measurement sites on each test rail.

Profiles were digitized and then processed to produce the dimensions shown in Figure 3 along with gross area (complete area above the lines projected from the fishing surfaces) and total head area (above projections from the fishing surfaces but within the original profile). Portions of these data have been subjected to two types of statistical analysis by independent organizations. The Association of American Railroads (AAR) made the more elaborate, and more rigorously correct, analysis of covariance by using all original data for decisions about statistical significance in which wear is described by a linear wear model. The Transportation Systems Center (TSC) of the U.S. Department of Transportation made the second, simpler, analysis. This analysis also used a linear wear model and determined the parameters of the model by regression techniques, but testing for statistical significance was applied only to the wear rates derived from the regression analysis and the original data were not used.

Results of both analyses are presented for rates of gage-face wear from the first experiment in section 3. The more elaborate method has also been used to study the head-height loss and head-area reduction indicators of wear in section 3 (first experiment). The simpler method has been applied alone to the rates of gage-face wear from section 13 (first experiment) and from section 3 (second experiment); at this time, results are available only for the high rail.

WEAR

The extremes of profile shape that occur after approximately 118 million Mg (130 million tons) in conditions of combined underlubrication and very generous lubrication are shown in Figure 3. The standard rail has a well-developed "front porch" formed by metal flowing down the gage face. This porch forms in the underlubricated regime. All of the other metallurgies are far less susceptible to this behavior than is standard rail.

The wear data on gage-face loss for all five metallurgies on the 1:40-cant tie plate in section 3 are shown in Figure 4. The scatter for standard and HiSi rails was substantially greater than that of the other metallurgies because there were about 10 different heats of standard and HiSi rail but only one or two different heats of the other metallurgies.

The wear rates for the three different measures of wear above and below the transition in lubrication and for the different tie-plate cants are summarized in Table 2. The results indicate that both tie-plate cant and metallurgy have a significant effect on all three measures. Wear rates above 41 million Mg (45 million tons) are substantially lower than those below 41 million Mg. At <41 million Mg, the 1:14 tie-plate cant yields about 20 percent higher wear rates for gage-face wear and head-area loss and the 1:40 cant produces higher rates of head-height loss. Typically, wear rates for HH and CrMo rail are lower than those for the other metallurgies. HH shows the least gage-face wear, but CrMo shows the least head-height loss. The degree of difference between the behavior of the different metallurgies is less above 41 million Mg than below that load, which does suggest the presence of a metallurgy-lubrication interaction.

To explore this last point further, figures of merit (FMs) were calculated for all the metallurgies in each lubrication regime to provide a quantitative average ranking of each metallurgy. The FM is the number of times better a premium rail wears (on average) than does standard rail tested under the same condition. The FMs are given in Table 3. This type of presentation reveals clearly that generous lubrication tends to decrease the advantage in wear resistance achieved by using a premium metallurgy.

The decrease is most marked for gage-face wear, where only HH rail seems significantly better than standard rail in the generously lubricated regime. In addition, the ranking of the metallurgies is not the same for head-area and gage-face loss. Although FHT rail is no better than HiSi rail in its resistance to gage-face loss, it is significantly better in terms of head-area loss.

Tables 4 and 5 illustrate that the agreement between the two different types of analysis is very good even if the transition traffic load is shifted slightly to 36 million Mg (40 million tons). Based on observed standard deviations, the FMs for gage-face wear have a tolerance of +25 to -15 percent.

The simpler TSC method of analysis is more easily implemented and was used, therefore, to test for position-in-curve effects and to provide a preliminary glimpse of the data from section 13 (first experiment) as well as those from section 3 (second experiment).

The magnitude of the position-in-curve effect in section 3 is illustrated by the data given in Table 6. In the underlubricated regime, the section 4 end of section 3 produced slightly higher wear rates on average than did either the middle or the section 2 end. In the more generously lubricated regime, however, the relative differences were greater, the highest wear occurring at the center of the curve. Again, the behavior was not strongly dependent on where the lubrication transition was selected. Table 7 summarizes the results of statistical testing for significance and shows that, indeed, the position-in-curve effect tended to increase at the expense of the primary metallurgy and cant effects as lubrication improved.

Rates of gage-face wear from section 13 (first experiment) are summarized in Table 8. The same general pattern was observed as that in section 3 except that a statistically significant position-in-curve effect was not noticed. HH rail gave the highest resistance to gage-face wear, and FHT rail was noticeably better than HiSi and standard rail. In the more generously lubricated regime above 36 million Mg (40 million tons), the behavior of all metallurgies was essentially the same.

Before the results from the second experiment in section 3 are discussed, it will be informative to comment briefly on the effect of heat-to-heat variations that occurred in the standard rail of the first experiment. There were four combinations of high-wear-rate heat and tie-plate cant. If these four combinations were removed, variations of the standard rail data set would be comparable to those of the premium metallurgies. For all three measures of wear, however, the removal of these combinations would reduce average wear rates by only 10 percent, and the FM would be reduced by a corresponding 10 percent.

Preliminary results on gage-face wear from the second experiment in section 3 are summarized in Table 9. The table below compares these results with results from the first experiment (1 cm/Mg = 0.358 in/ton):

Location	Gage-Face Wear (cm/million Mg)	
	First Experiment	Second Experiment
Section 2 end	0.003-0.004	0.001 25
Middle	0.0044-0.005	0.001 37
Section 4 end	0.0014-0.0022	0.001 03

The results in terms of FM are in close agreement with those from the generously lubricated regime of the first experiment except that the CrMo rail performed somewhat better than the HH rail. The reader must exercise some caution in deciding whether the difference in FM for CrMo and HH is real because it was only in the section tion 2 end that both appeared to be appreciably different.

Overall, the observed wear rates in the second experiment were only one-third to one-half the magnitude of those observed in the first experiment.

Table 10 summarizes the pertinent statistical information. Tie-plate cant did not appear to be statistically significant, although the position-in-curve effect did appear to be statistically significant when tie-plate cant was included in the analysis. If tie-plate cant was removed as a variable, the statistical strength of the position-in-curve effect weakened.

It will be informative to consider how these results compare with those from other investigations. Figure 5 shows the range of wear rates (head-area loss) for the metallurgies tested in section 3 (5° curve) relative to those reported by others (1-3). The data of Hay and

Figure 4. Gage-face wear of all five metallurgies on 1:40-cant tie plates in section 3.

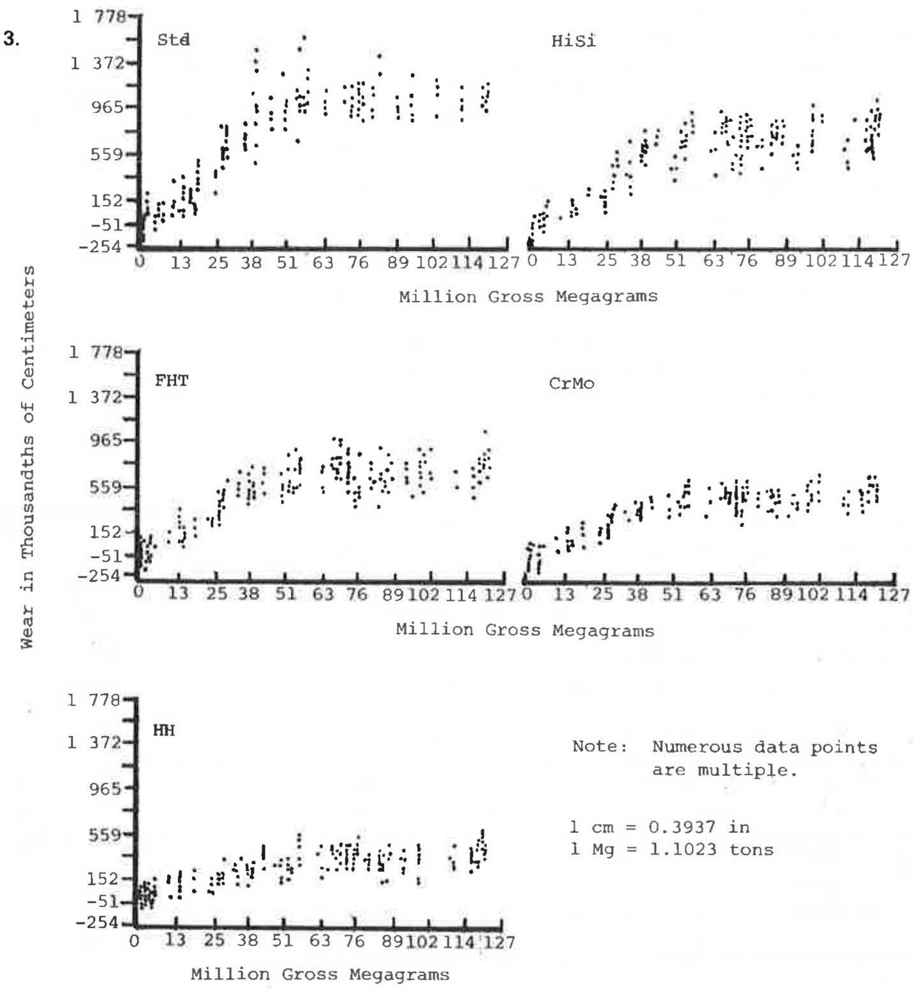


Table 2. Wear rates above and below the 41 000 000-Mg lubrication transition for various tie-plate cants.

Wear	Tie-Plate Cant	HH		HiSi		FHT		CrMo		Standard	
		Below	Above	Below	Above	Below	Above	Below	Above	Below	Above
Gage-face wear (cm/ million Mg)	1:40	0.007 57	0.000 83	0.016 17	0.000 86	0.015 36	0.002 29	0.009 91	0.002 01	0.022 59	0.002 01
	1:14	0.010 72	0.002 51	0.019 75	0.004 16	0.018 38	0.003 10	0.012 40	0.003 77	0.023 32	0.003 77
	1:30	0.008 32	0.001 51	0.014 33	0.002 71	0.015 31	0.002 85	0.011 20	0.002 76	0.021 73	0.002 76
Head-area loss (cm^2/million Mg)	1:40	0.005 49	0.003 28	0.058	0.007 85	0.050 28	0.003 28	0.032 07	0.006 57	0.091 93	0.011 93
	1:14	0.003 67	0.006 57	0.064 78	0.008 5	0.047 64	0.010 64	0.030 21	0.007 64	0.088 93	0.009 07
	1:30	0.003 03	0.006 93	0.050 57	0.011	0.044 78	0.013 07	0.034 43	0.066 07	0.087 78	0.011 14
Head-height loss (cm/ million Mg)	1:40	0.003 54	-[a]	0.004 02	-[a]	0.002 71	-[a]	0.002 04	-[a]	0.007 15	0.000 03
	1:14	0.003 04	-[a]	0.003 74	-[a]	0.001 56	0.000 14	0.001 48	-[a]	0.006 51	0.000 31
	1:30	0.002 51	-[a]	0.003 41	-[a]	0.002 20	-[a]	0.001 45	0.000 16	0.006 23	0.000 83

Note: 1 cm/Mg = 0.358 in/ton; 1 cm^2/Mg = 0.14 in^2/ton.
[a]No significant wear.

Table 3. Figures of merit above and below the 41 000 000-Mg lubrication transition for various tie-plate cants.

Wear	Metallurgy	1:40 Cant Below	1:40 Cant Above	1:14 Cant Below	1:14 Cant Above	1:30 Cant Below	1:30 Cant Above	Avg Below	Avg Above
Gage-face wear	HH	3.0	2.4	2.2	1.5	1.8	1.8	2.6	1.9
	HiSi	1.4	1.3	1.2	0.9	1.0	1.0	1.4	1.1
	FHT	1.5	0.9	1.3	1.2	1.0	1.0	1.4	1.0
	CrMo	2.3	1.2	1.9	1.2	1.0	1.0	2.0	1.2
	Std	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Head-area loss	HH	2.6	3.6	2.4	1.4	2.9	1.6	2.6	2.2
	HiSi	1.6	1.5	1.4	1.1	1.7	1.0	1.6	1.2
	FHT	1.8	3.6	1.9	0.8	2.0	0.8	1.9	1.8
	CrMo	2.9	1.8	2.9	1.2	2.5	1.8	2.9	1.6
	Std	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Head-height loss	HH	2.0	-[a]	2.1	-[a]	2.5	-[a]	2.2	-[a]
	HiSi	1.8	-[a]	1.7	-[a]	1.8	-[a]	1.8	-[a]
	FHT	2.6	-[a]	4.2	-[a]	2.8	-[a]	3.2	-[a]
	CrMo	3.5	-[a]	4.9	-[a]	4.3	-[a]	4.2	-[a]
	Std	1.0	-[a]	1.0	-[a]	1.0	-[a]	1.0	-[a]

Note: Figures of merit = standard carbon wear rate ÷ premium metallurgy wear rate on specific tie-plate cant.
[a]No significant wear.

Table 4. Average gage-face loss for section 3: comparison of results of AAR and TSC analyses.

Metallurgy	Below Lubrication Transition: AAR (<41 000 000 Mg) Gage-Face Wear (cm/million Mg)	FM	TSC <41 000 000 Mg Gage-Face Wear (cm/million Mg)	FM	TSC <37 000 000 Mg Gage-Face Wear (cm/million Mg)	FM	Above Lubrication Transition: AAR (>41 000 000 Mg) Gage-Face Wear (cm/million Mg)	FM	TSC >41 000 000 Mg Gage-Face Wear (cm/million Mg)	FM	TSC >37 000 000 Mg Gage-Face Wear (cm/million Mg)	FM
HH	0.0089	2.6	0.0086	2.8	0.0089	2.8	0.0016	1.9	0.0022	1.4	0.0025	1.8
HiSi	0.0167	1.4	0.0167	1.4	0.0178	1.4	0.0025	1.1	0.0033	0.9	0.0042	1.1
FHT	0.1620	1.4	0.0159	1.5	0.0170	1.5	0.0028	1.0	0.0025	1.2	0.0036	1.3
CrMo	0.0112	2.0	0.0114	2.1	0.0117	2.2	0.0025	1.1	0.0030	1.0	0.0036	1.3
Standard	0.0226	1.0	0.0243	1.0	0.0254	1.0	0.0028	1.0	0.0030	1.0	0.0044	1.0

Note: 1 Mg = 1.1 tons; 1 cm = 0.3937 in.

Table 5. Tie-plate-cant effect for section 3.

Cant	Gage-Face Wear (cm/million Mg): Below Lubrication Transition: AAR (<41 000 000 Mg)	TSC <41 000 000 Mg	TSC <37 000 000 Mg	Above Lubrication Transition: AAR (>41 000 000 Mg)	TSC >41 000 000 Mg	TSC >37 000 000 Mg
1:40	0.0142	0.0145	0.0156	0.0015	0.0022	0.0031
1:30	0.0142	0.0145	0.0156	0.0025	0.0031	0.0036
1:14	0.0170	0.0173	0.0178	0.0033	0.0033	0.0044

Note: 1 cm = 0.3937 in; 1 Mg = 1.1 tons.

others (1) are shown as average curves. FAST data are consistent with the trend of results reported by Curcio and others (2) for unit-train-type operations in Australia. FAST wear in the underlubricated region was substantially more severe than that reported by Hay and others for more general types of railroad service in the United States. Even wear reported by Rougas (3) for heavy unit train service on the Bessemer and Lake Erie Railroad was less severe than FAST wear.

The expression proposed by Kalousek and Bethune (4) for volumetric wear can be recast into the following form:

$$V_1/V_2 = (C_1/C_2) \cdot (H_2^{\alpha_2}/H_1^{\alpha_1}) \tag{1}$$

where

V = volumetric wear,
H = hardness,
1 and 2 = different metallurgies, and
C and α = empirical constants.

The presumption is that lateral force, lateral and vertical creep, and the angle of the gage face to the lateral force vector are not functions of metallurgy. Thus, the ratio V_1/V_2 is really an FM if V_1 is taken to represent standard rail. If volumetric wear is thought to be best represented by head-area loss, the FAST FM can be plotted versus the gage-face hardness ratio, as shown in Figure 6.

Two different linear plots (α_1, α_2 = 1) appear to obtain for the premium rails—one for heat-treated rails and the other for alloy rails. Furthermore, the slopes of the lines appear to be a function of lubrication and the heat-treated rails to be less influenced by lubrication than alloy rails. Whether the gage-face-wear results for the CrMo rail in the second experiment contradict this will not be known until the data on head-area loss are analyzed and hardnesses are taken on the gage face.

RAIL AND WELD FAILURE

The very low wear rates of standard rail observed under conditions of generous lubrication would project to a total rail life (on a 5° curve) between 680 million Mg (750 million tons) (first experiment results) and 1.134 billion Mg (1.25 billion tons) (second experiment results) for a 19-mm (0.75-in) gage-face-wear condemning limit. However, long before these traffic loads were reached, appreciable amounts of rail would have to be replaced because of fatigue. Indeed, FAST has been a prodigious generator of fatigue failures, in both rail and weldments, in the well-lubricated regime. Table 11 summarizes the total number of weld- and head-type failures that have occurred to 227 million Mg (250 million tons) in the first and second metallurgy experiments.

In the first experiment, 24 percent of the plant welds in section 3 and 27 percent in section 13 failed. Although only 22 (section 3) and 10 (section 13) field welds (thermite) were placed in the original construction, the total number of field weld failures exceeded these numbers because replacement welds also failed. These weld failures, along with rapid wear in the underlubricated regime, necessitated the rerailing of sections 3 and 13 at 122 million Mg (135 million tons). However, limited traffic load was accumulated in the first experiment, and relatively few head-type defects occurred; of those defects, three each were in FHT and standard rail, two were in HH rail, and one was in HiSi rail.

In the second experiment, less wear and greater load resulted in the generation of more head-type defects. In addition, redesign of the experiment configuration reduced the number of plant weld failures to 8 percent in both sections 3 and 13. Although the number of field weld failures was reduced, it still remained at an inconveniently high level.

Not all of the rail defects should be considered true fatigue defects. Since four of the head defects that occurred in section 3 were at or very near the ends of metallurgy segments, they can be related to mechanical joint assemblies or maintenance problems or both. If these four defects were removed from consideration, along with five head defects associated with an apparently very dirty heat of standard rail in section 13 (5), the overall defect rates would drop to 12 defects/km (20 defects/mile) for section 3 and 6 defects/km (10/defects mile) for section 13. These rates would be relatively un-

Table 6. Position-in-curve effect for section 3.

Location	Gage-Face Wear (cm/million Mg) Below Lubrication Transition <41 000 000 Mg	Below Lubrication Transition <37 000 000 Mg	Above Lubrication Transition >41 000 000 Mg	Above Lubrication Transition >37 000 000 Mg
Section 2 end	0.0145	0.0148	0.0028	0.0039
Middle	0.0148	0.0156	0.0044	0.0050
Section 4 end	0.0170	0.0187	0.0014	0.0022

Note: 1 cm = 0.3937 in; 1 Mg = 1.1 tons.

Table 7. Results of statistical tests for significance for section 3: gage-face-wear rates for various tie-plate cants and positions in curve.

Lubrication Transition (Mg)	Effect or Interaction	Observed F	99 Percent Level F Required	99 Percent Level Significant	95 Percent Level F Required	95 Percent Level Significant
<37 000 000	Cant	7.98	6.23	Yes	3.63	Yes
	Position in curve	16.79	6.23	Yes	3.63	Yes
	Metallurgy	90.96	4.77	Yes	3.01	Yes
	Cant position in curve	3.73	4.77	No	3.01	Yes
	Position in curve/metallurgy	1.27	3.89	No	2.59	No
	Metallurgy/cant	0.75	3.89	No	2.59	No
>37 000 000	Cant	3.86	6.23	No	3.63	Yes
	Position in curve	57.97	6.23	Yes	3.63	Yes
	Metallurgy	14.85	4.77	Yes	3.01	Yes
	Cant/position in curve	4.57	4.77	No	3.01	Yes
	Position in curve/metallurgy	0.96	3.89	No	2.59	No
	Metallurgy/cant	2.19	3.89	No	2.59	No

Note: 1 Mg = 1.1 tons.

Table 8. Gage-face-wear results for section 13.

Lubrication Transition (Mg)	Metallurgy	Gage-Face Wear by Position in Curve (cm/million Mg) A	B	C	D	Avg	Avg FM
<37 000 000	HH	0.0058	0.0047	0.0070	0.0058	0.0061	3.5
	HiSi	0.0145	0.0137	0.0125	0.0120	0.0131	1.6
	FHT	0.0106	0.0106	0.0095	0.0103	0.0103	2.1
	Standard	0.0176	0.0257	0.0262	0.0167	0.0215	1.0
	Avg	0.0123	0.0137	0.0139	0.0128		
>37 000 000	HH	0.0039	0.0033	0.0008	0.0028	0.0028	1.3
	HiSi	0.0070	0.0003	0.0016	0.0039	0.0033	1.1
	FHT	0.0056	-0.0003	0.0028	0.0033	0.0028	1.3
	Standard	0.0044	0.0028	0.0028	0.0044	0.0036	1.0
	Avg	0.0053	0.0017	0.0019	0.0036		

Note: 1 cm/Mg = 0.358 in/ton; 1 Mg = 1.1 tons.

Table 9. Preliminary gage-face-wear results for second tests on section 3.

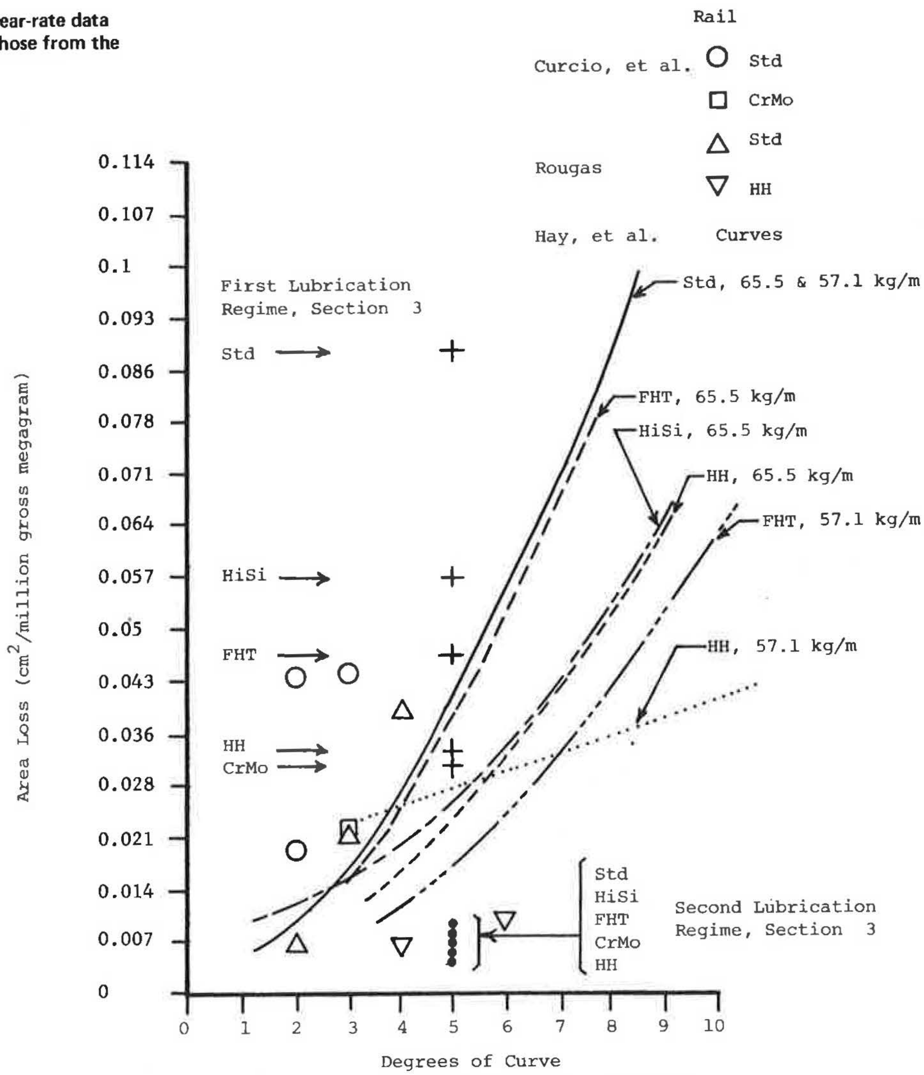
Metallurgy	Gage-Face Wear by Location (cm/million Mg) Section 2 End	Middle	Section 4 End	Avg[a]	FM
HH	0.0011	0.0008	0.0005	0.0008	2
HiSi	0.0016	0.0019	0.0014	0.0016	1
FHT	0.0014	0.0016	0.0014	0.0014	1.2
CrMo	0.0003	0.0005	0.0005	0.0005	3
Standard	0.0016	0.0019	0.0014	0.0016	1

Note: 1 cm = 0.3937 in; 1 Mg = 1.1 tons.
[a] Rounded off.

Table 10. Results of statistical tests for significance of section 3 gage-face-wear rates: second experiment.

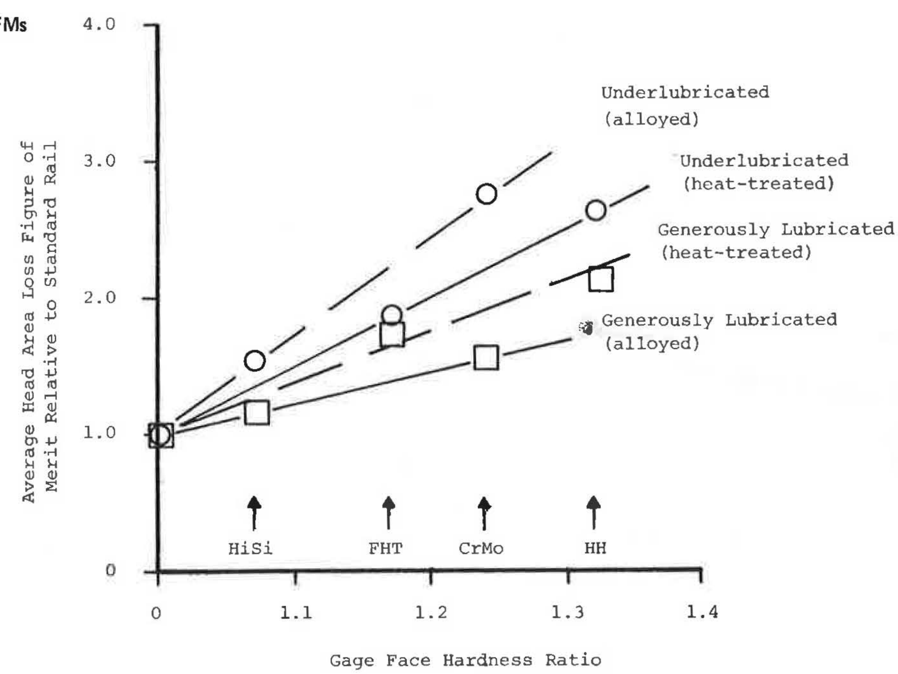
Presence of Cant Effect	Effect	Observed F	99 Percent Level F Required	99 Percent Level Significant	95 Percent Level F Required	95 Percent Level Significant
Present	Metallurgy	71.496	9.15	Yes	4.53	Yes
	Position in curve	11.806	10.9	Yes	5.14	Yes
	Tie-plate cant	3.791	10.9	No	5.14	No
Removed	Metallurgy	42.115	7.01	Yes	3.84	Yes
	Position in curve	6.995	8.65	No	4.46	Yes

Figure 5. Comparison of wear-rate data from various sources with those from the first FAST experiment.



Note: 1 cm^2/million Mg = 0.14 in^2/million tons; 1 kg/m = 1.98 lb/yd.

Figure 6. Gage-face hardness versus FMs for head-area loss.



biased for head defects to 227 million Mg (250 million tons).

Figure 7 shows the location by metallurgy and tie-plate cant for plant weld and head-type defects in the high rail of section 3. Without counting defects near segment ends, standard rail had five head-type fatigue defects, for a rate of 27 defects/km (44 defects/mile) of rail. HiSi rail was next, with only two defects for a rate of 11 defects/km (18 defects/mile). HH rail had only one head defect that was not at a segment end or weldment, which yielded a defect rate of 5.5 defects/km

Table 11. Total rail failures in metallurgy test sections 3 and 13.

		Plant Welds		Field Welds		Head Defects	
Experiment	Section	Initially Installed	Failed	Initially Installed	Failed[a]	Total	Rate (defects/km)
1[b]	3	180	44	22	44	2	
	13	64	17	10	17	7	
2[c]	3	120	10	0	12	15	17[d]
		48	4	0	8	7	6[e] or 18.6[d]

Note: 1 km = 0.62 mile.
[a]Includes replacement welds that also failed.
[b]Traffic load of 123 million Mg (135 million tons).
[c]Traffic load of 227 million Mg (250 million tons).
[d]Indicates defect rate at 142 million Mg/km (250 million tons/mile) of track.
[e]Defect rate adjusted to eliminate failures and rail length associated with "dirty" standard rail.

Figure 7. Occurrence of weld and head failures in high rail of section 3.

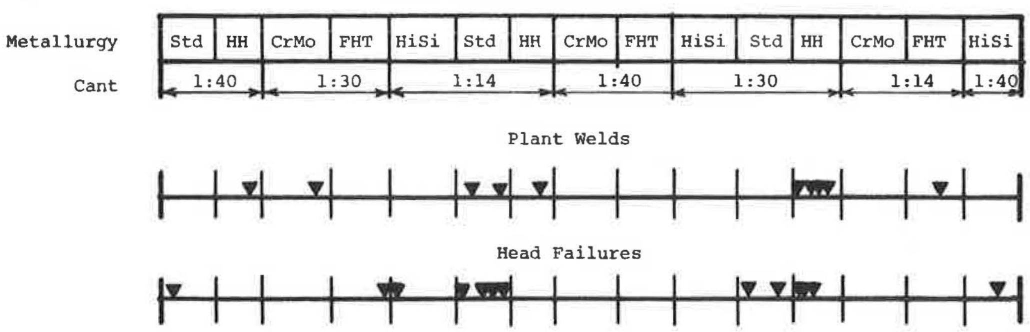
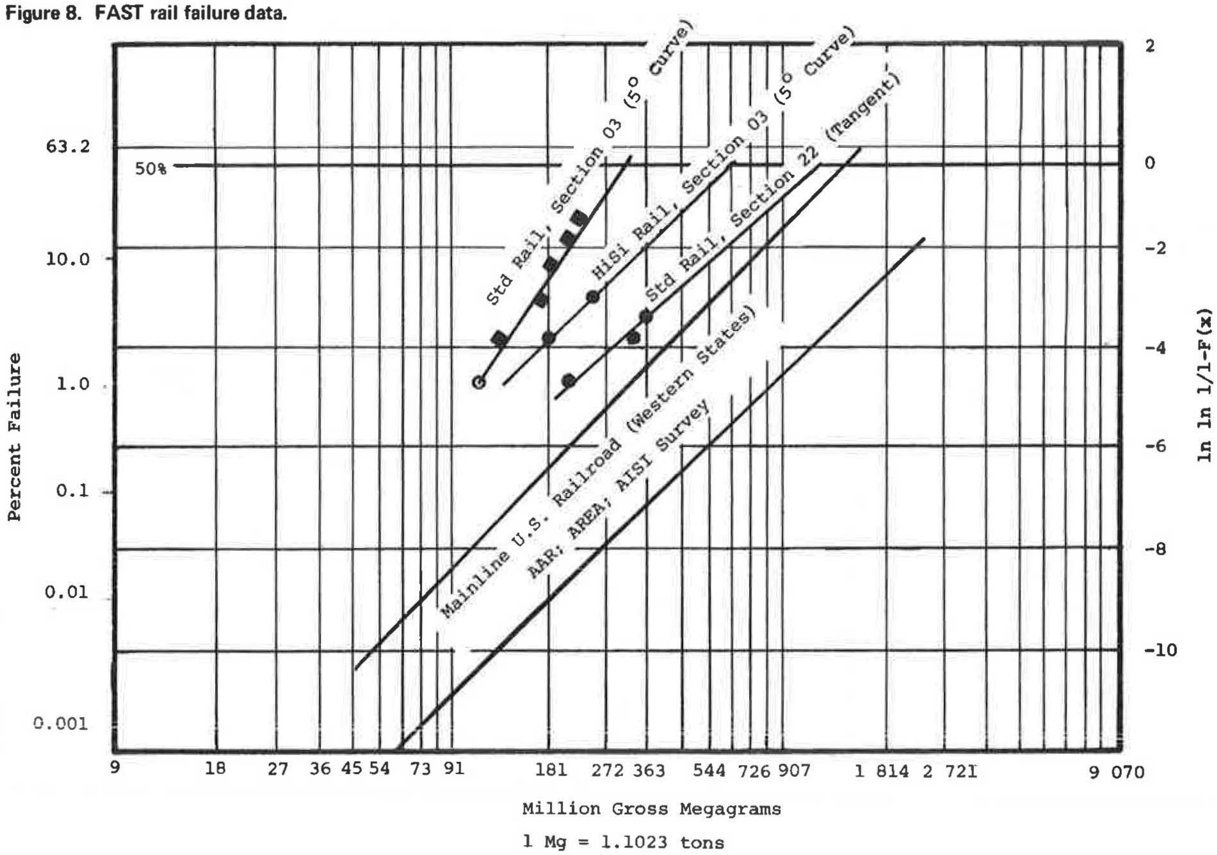


Figure 8. FAST rail failure data.



(9 defects/mile). FHT rail had only one head defect, and it was at a segment end.

HH rail had the most plant weld failures—six. Standard rail had two plant weld failures, and CrMo and FHT rail had only one each within the metallurgy test section. In section 13, standard rail had two plant weld failures, and FHT and HiSi rail had one failure each.

The 1:14 tie-plate cant was associated with 9 failures (4 plant weld and 5 head defect), the 1:30 cant with 10 failures (5 plant weld and 5 head defect), and the 1:40 cant with 3 failures (1 plant weld and 2 head defect). However, some of these did occur near segment ends and may therefore be considered suspect. In addition, the experiment design placed the 1:14 and 1:30 cants in the midregion of the curve whereas the 1:40 cant was positioned at the ends of the curve under standard, HH, and HiSi rails.

The relation between rail failure and traffic load is shown in Figure 8 for two of the rail metallurgies tested in section 3. The behavior of standard rail in FAST section 22 (continuous welded rail, tangent track) and that of rail in some western U.S. main-line service (6) are also shown for comparison. In section 3 (5° curvature), the percentage failure of standard rail at 181 million Mg (200 million tons) has been approximately 10 times that of standard rail in section 22 and more than 100 times that of the average of rail in some western U.S. main-line service. HiSi rail has a slightly lower failure rate than standard rail. There were not enough fatigue failures in FHT, HH, and CrMo rail to permit any quantitative assessment of how much better they might be than standard rail. Although the rail population of each metallurgy is small and a larger sampling might have produced less extreme behavior, the heavier wheel loads and unit train character of the FAST operation, along with the relatively high portion of curved track, have contributed to a substantial increase in the rate of rail fatigue failure.

Typically, both plant and field welds have a nearly horizontal web crack that extends to either side of the weld region. However, the plant weld failure originates from a transverse vertical crack that develops at the edge of the shear drag region under the gage side head-web fillet. A substantial period of vertical fatigue growth [~18 million Mg (~20 million tons)] frequently occurs before the crack turns horizontal in the web. On the other hand, the field weld failure sometimes appears to be initiated in the heat-affected zone, the crack then propagating through the weld itself. Detail fractures initiate at depths near 9.5 mm (0.375 in) below the gage corner just beneath the heavily cold-worked surface region. The growth of the transverse crack develops clearly demarked growth rings because of the change in train direction each day [~0.9 million Mg (1 million tons)]. The period in which the crack grows from the 10 percent size [10-mm (0.4-in) diameter] to final fracture at a radius of 40.6 mm (1.6 in) is typically between 9 million and 13 million Mg (10 million and 15 million tons).

CONCLUSIONS

The wear tests in the first and second metallurgy tests have shown that HH and CrMo rail exhibited the best resistance to wear in the FAST loading environment. There was a strong interaction between lubrication and metallurgy and, under conditions of generous lubrication, the premium metallurgies did not appear to benefit as much from lubrication as did standard rail.

In the underlubricated wear regime, the 1:14 tie-plate cant produced about 20 percent more gage-face and head-area-loss wear than did the other cants. However, the 1:40 cant produced somewhat greater head-height loss. The tie-plate-cant effect was diminished considerably under conditions of generous lubrication, and position-in-curve effects depended on the level of lubrication (they reversed in character as lubrication level varied from one extreme to another).

When conditions of generous lubrication served to extend rail life substantially, fatigue failure—both in the head of the rail and in weldments—became the dominant failure mode. At 227 million Mg (250 million tons) of traffic, the true FAST head-defect rate was near 6-12 defects/km (10-20 defects/mile). Substantially more rail and plate weld failures were associated with the 1:14 and 1:30 cants than with the 1:40 cant. The fatigue failure rates of standard rail in track with a 5° curve and in tangent track were approximately 100 and 10 times, respectively, that reported for rail in some western U.S. main-line service.

ACKNOWLEDGMENT

We wish to express our appreciation to N. Parikh, W. Bagnuollo, S. Roberts, and B. Pearson of the AAR staff and to J. Morris of the TSC staff for their continued help in obtaining the data needed for analysis. In addition, a very enlightening review of the wear information by W. Pellini is gratefully acknowledged.

REFERENCES

1. W. W. Hay, A. J. Reinschmidt, P. T. Bakas, and P. M. Schuch. Economic Evaluation of Special Metallurgy Rails. Univ. of Illinois, Urbana, Rept. Eng. 76-2002, Jan. 1976. NTIS: PB 252 024.
2. P. Curcio, S. Marich, and G. Nisich. Performance of High-Strength Rails in Track. Heavy-Haul Railways Conference, Perth, Australia, Session 313, Paper I.10, Sept. 1978.
3. M. Rougas. Observations on the Effect of Heavy Wheel Loads on Rail Life. Federal Railroad Administration, U.S. Department of Transportation, Rept. FRA-ORD-76-243, Oct. 1975.
4. J. Kalousek and A. E. Bethune. Rail Wear Under Heavy Traffic Conditions. Department of Research, Canadian Pacific, Ltd., Montreal, 1978.
5. L. D. Fleming and M. J. Wisnowski. Investigation of a Failed Rail from FAST. Assn. of American Railroads Tech. Center, Chicago, Rept. R-371, May 1979.
6. D. H. Stone. Track-Train Dynamics Contributions to Rail Metallurgy. AREA Bull., Vol. 80, June-July 1979.

Publication of this paper sponsored by Committee on Track Structure System Design.

Deformation Behavior of Rail Steels

D. H. Stone, S. Marich, and C. M. Rimnac

The cyclic deformation behavior of three rail steels was determined under conditions of uniaxial plane-strain compression. Two loading programs were used: (a) one load (simple loading) for the entire test and (b) two loads (split loading) in which the load was increased at set intervals during the test. The results for simple loading showed that the steel softened under cyclic compression; i.e., for a constant stress, compressive cyclic loading caused an increase in strain. Increasing the applied stress increased both the rate and the amount of softening. Rails with higher hardness and yield strength showed an increase in deformation resistance. Split loading produced either increased or decreased resistance to deformation, depending on the type of steel. An equation is presented that can be used to predict the expected amount of plastic flow in rail in service.

The deformation behavior of rail steels has taken on great importance with the increasing severity of service conditions. The average weight of a carload has risen 20 percent in 10 years, and train speeds have also increased (1). Consequently, rails are wearing and failing at higher than expected rates. Research into this problem is being done by several agencies, including Battelle Memorial Institute and the Association of American Railroads (AAR).

Research at Battelle showed that fully reversed cyclic straining caused softening of standard rail steel at low strain levels and hardening at strain levels greater than 0.6. However, when the steel was cycled with a mean tensile or compressive strain, softening occurred over the entire strain range. In addition, the softening rate, or relaxation rate, was the same for both mean strains. The results of the Battelle studies indicated that tests made with fully reversed strain amplitudes may not accurately duplicate the straining of rail in service (2).

Because rails undergo compressive loading, Marich and Curcio (3) proposed that deformation tests on rails should be made in compression, thus eliminating the Bauschinger effect and approximating more closely actual service conditions. Their work on rails tested in monotonic plane strain compression showed that, as the yield strength of the rail increased, the depth of deformation in the railhead decreased, thus decreasing the occurrence of shelling and transverse defects. In addition, the work-hardening behavior of a hot-rolled pearlitic steel was related to dislocation processes occurring in the ferrite lamellae, a decrease in the interlamellar spacing, and dislocation tangles in the cementite.

As an initial part of this study, the deformation pattern and microstructure of rails that had undergone 468 million-662 million gross Mg (515 million-728 million gross tons) of traffic were evaluated to characterize service-induced deformation behavior. The resulting hardness profiles and microstructures serve as standard to ensure that laboratory experiments are reproducing service conditions.

This present investigation is part of the AAR's effort to increase research and development on the improvement of rail performance. The purpose of this project is to study the deformation behavior of several rail steels under cyclic uniaxial plane strain compression.

OBSERVED BEHAVIOR IN RAILS REMOVED FROM SERVICE

Three rails removed from service on the Union Pacific Railroad were used to characterize rail work hardening attributable to plastic flow and the change in microstructure that accompanies deformation. Two of the rails had undergone 468 million gross Mg (515 million gross tons) of traffic, and one of the rails had undergone 662 million gross Mg (728 million gross tons) of traffic. Figure 1 shows the results of two Vickers microhardness test surveys made at the gage corner of the rail. The rails typically had been work hardened to 85-95 Vickers hardness above the base hardness [Vickers microhardness numbers are approximately equivalent to Brinell hardness numbers (BHN)]. It is important to note that, between 6 and 8 mm (0.24 and 0.31 in) in depth, a zone of work-softened material exists. It has been shown, by Leis (2) for rail steel and Park and Stone (4) for wheel steels, that these pearlitic steels work soften under cyclic strains of less than 0.6 percent.

There are also dramatic changes in the microstructure that may be associated with work hardening. Figures 2-4 show scanning electron microscope photomicrographs of the same rail specimen, after 662 million gross Mg (728 million gross tons) of service, at depths of 2.25, 6.75, and 7.5 mm (0.09, 0.27, and 0.30 in), respectively. At 2.25 mm (Figure 2), the material has been work hardened to 320 BHN and exhibits a very heavily deformed microstructure within which the cementite plates either have become kinked (in a wavelike pattern) and cracked or have thinned out. The difference in deformation behavior could be associated with the orientation of the cementite plates within individual cells relative to the applied load. In the work-softened zone, the cementite is either straight with some cracking or slightly deformed in a sinusoidal pattern. In the base material at 7.5 mm (Figure 4), the cementite plates are in their normal straight and undeformed condition. Several investigators have observed the same microstructure as that shown in Figure 2 in cold-drawn, high-carbon steel wire (5, 6).

TESTING PROCEDURE

Materials

Three rail steels were tested: (a) hot-rolled carbon steel, (b) heat-treated carbon steel, and (c) pearlitic chromium-molybdenum (CrMo) alloy. Their compositions and mechanical properties are given in Table 1.

The hot-rolled carbon steel had a fully pearlitic microstructure. The heat-treated carbon steel was also pearlitic but had a smaller grain size. The alloyed pearlitic rail was also fine grained.

Samples were cut from the railhead with the deformation face parallel to the running surface (Figure 5). The samples were 6.35 mm (0.25 in) thick and, except for the heat-treated samples, which had a width of 23.8 mm (0.9375 in), were 25.4 mm (1 in) wide.

Because the hardness of rails varies with depth from the surface, the average hardness of each sample was determined and any sample that varied more than 2 points Rockwell C from the average was discarded. The hardness values given in Table 1 are thus the average of the samples tested.

Mechanical Testing

The compression test used in this project was designed by Watts and Ford (7) for testing steel sheet and strip. The test consists of applying a compressive load to the sample by means of two parallel indenting dies (see Figure 6). The width b of the dies was equal to the thickness t of the samples. It has been found that the true yield stress is achieved only when t is an integral multiple of b. By keeping b small in comparison with sample width w, the deformed region of the sample is constrained in the width dimension by the undeformed material on either side. Thus, the sample deformed under plane strain conditions. This test approximates the loading conditions experienced by rail in service, especially in tangent track and on the low rail of curves.

Figure 1. Hardness profiles of standard carbon steel rail in the high rail of a 1° curve after 468 million gross Mg (515 million gross tons) of service.

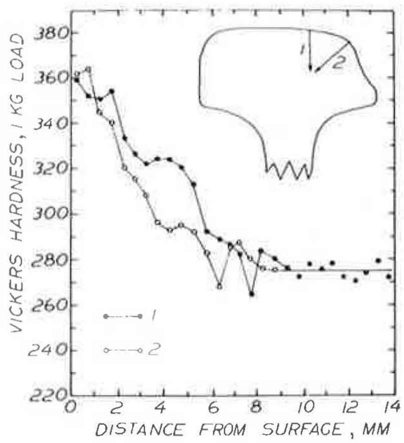


Figure 2. Microstructure of rail specimen 2.25 mm (0.9 in) below running surface (4900X etched in Nital).

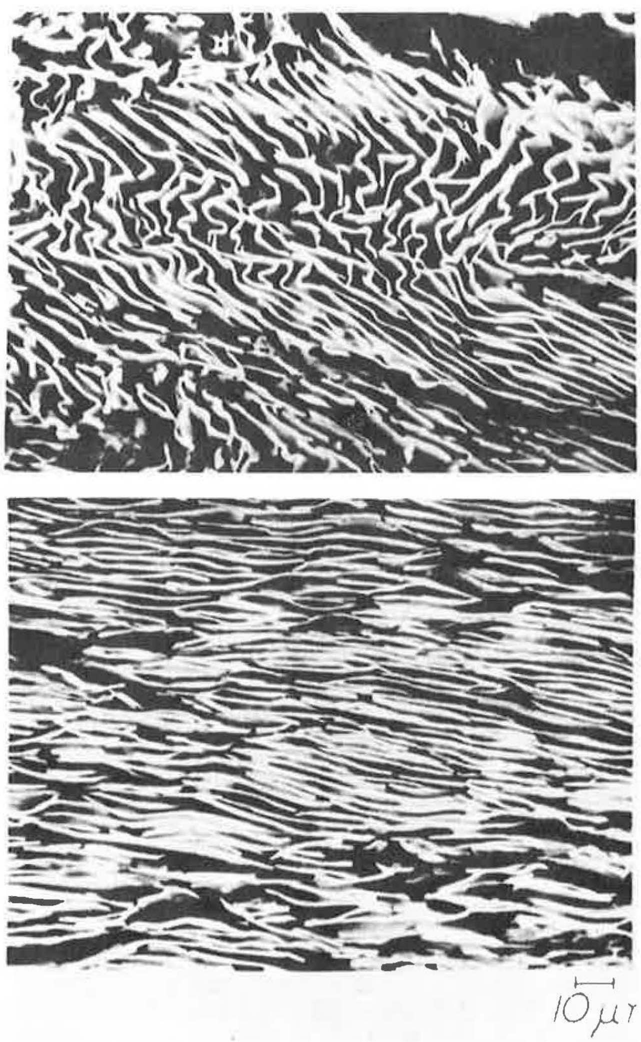


Figure 3. Microstructure of rail specimen 6.75 mm (0.27 in) below running surface (4900X etched in Nital).

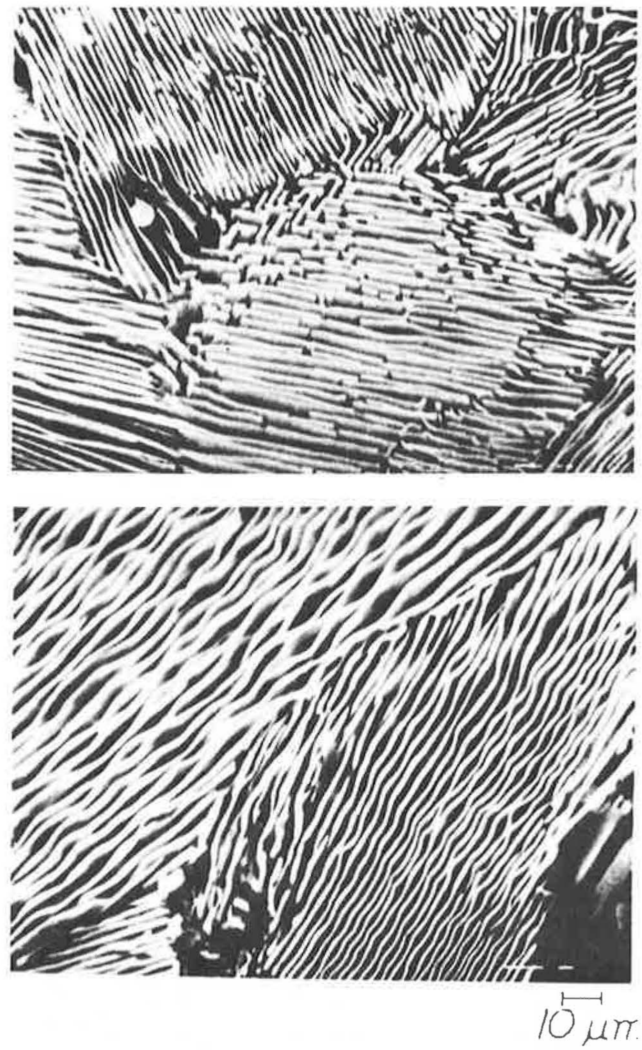


Figure 4. Microstructure of rail specimen 7.5 mm (0.3 in) below running surface (4900X etched in Nital).

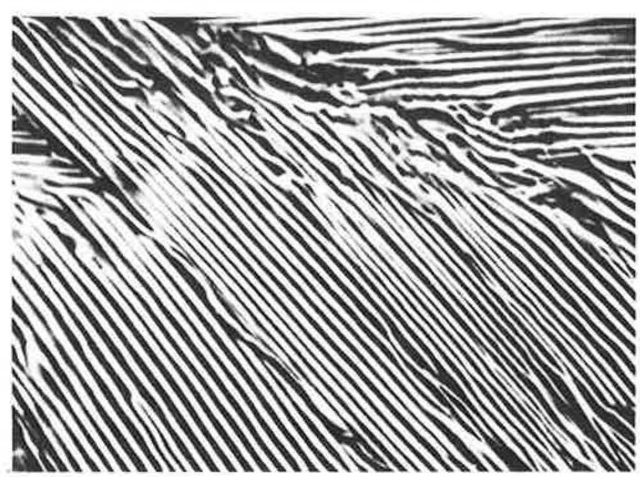


Table 1. Composition and mechanical properties of rail steels.

Type of Rail Steel	Composition (%)	Yield Strength (MPa)	Rockwell C Hardness
As-rolled carbon steel	0.69-0.82 carbon, 0.7-1.0 manganese, 0.04 max phosphorus, 0.05 max sulfur, 0.1-0.25 silicon	517	22-23
Heat-treated carbon steel	0.69-0.82 carbon, 0.7-1.0 manganese, 0.04 max phosphorus, 0.05 max sulfur, 0.1-0.25 silicon	827	38-39
CrMo steel, pearlitic	0.78 carbon, 0.84 manganese, 0.22 silicon, 0.72 chromium, 0.19 molybdenum, 0.026 phosphorus, 0.022 sulfur	752	35-36

Note: 1 MPa = 145 lbf/in².

Figure 5. Test sample as taken from the head of a rail.

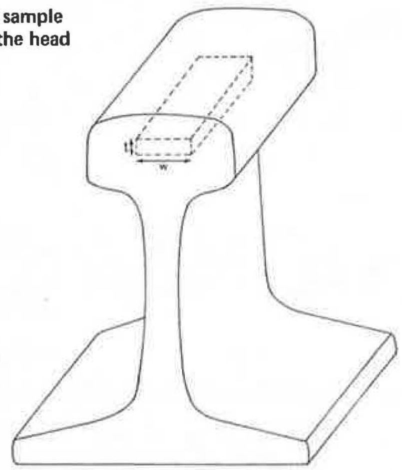
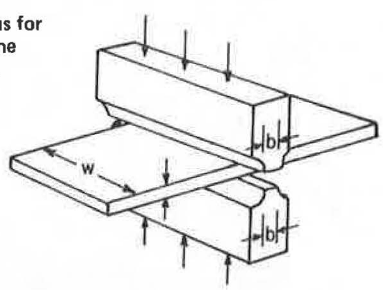


Figure 6. Apparatus for testing uniaxial plane strain compression.



Testing was done on a Material Testing System electrohydraulic close-loop machine in load control (cyclic deformation tests are usually run in strain control, but in this case load control was the simpler mode).

The ideal loading is zero to P_{max} (compressive). However, because of the instability of the samples at zero load, samples were loaded from -13 kN (-3000 lbf) to P_{max}. Loads were applied following a sine wave function at frequencies from 6 to 18 Hz. P_{max} varied from 110 to 200 kN (25 000-45 000 lbf).

To reduce friction effects, the contact surface of the specimens was covered with Teflon tape. In order to record the progressive deformation behavior, the reduction in sample thickness was measured by a micrometer to the nearest 0.002 54 mm (0.0001 in) after a set number of cycles. To facilitate plotting on logarithmic paper, thickness was measured after 1, 2, 5, 10, 20, 50, and 100 cycles, up to 100 000 cycles or 10 percent reduction in thickness, whichever came first. After each measurement, the surface was again covered with Teflon tape. In replacing the samples, care was taken to align the indentation with the dies. The load was then applied again for the next cyclic increment.

Two loading patterns were used. The first pattern (simple loading) consisted of cycling at the same load for the entire run. Simple loading was done for a range of loads to observe the effect of increasing stress on deformation behavior. The second loading pattern (split loading) consisted of cycling the sample at one load for 100, 1000, or 10 000 cycles and then finishing the run at a higher load. Split-loading tests were run after the simple-loading tests so that the low loads could be chosen for little or no cyclic deformation and the high loads for marked cyclic softening behavior. One split-loading set (three runs) was done for each rail type.

Calculations

The axial compressive stress for plane strain compression in this case is simply the applied load divided by the area being deformed:

$$\sigma = P/wb \quad (1)$$

Each measurement of sample reduction was converted to true strain by the following formulas (derived from the Von Mises yield criterion): For percentage reduction in thickness,

$$(t_0 - t)/t_0 = \epsilon_\epsilon \quad (2)$$

For plane strain,

$$\ln(1 - \epsilon_\epsilon) = \epsilon_\rho \quad (3)$$

For true strain,

$$(2/\sqrt{3})\,\epsilon_\rho = \epsilon_t \quad (4)$$

Graphs of true strain versus cycles were thus obtained for the rail steels for simple and split loading.

TESTING QUALIFICATIONS

Frequency Effect

The frequency of the wheels of a train going over a section of rail is about 3 Hz. Tests previously run at AAR were conducted at a low frequency (6 Hz) to approximate service conditions. Because of time limitations, however, the current tests were conducted at higher frequencies. The effect of frequency on strain was therefore examined.

The curves produced by progressive cyclic loading of hot-rolled carbon steel rail at 6, 12, and 18 Hz and at 827 MPa [120 000 lbf/in² (120 kips/in²)] are shown in Figure 7. From these curves, it was concluded that there was no significant frequency effect and the tests could be run at 18 Hz. This result was not unexpected. In this range, steels generally do not show a frequency effect. If the magnitude of the strain range were great,

Figure 7. Effect of frequency on compressive cyclic softening of rail steel.

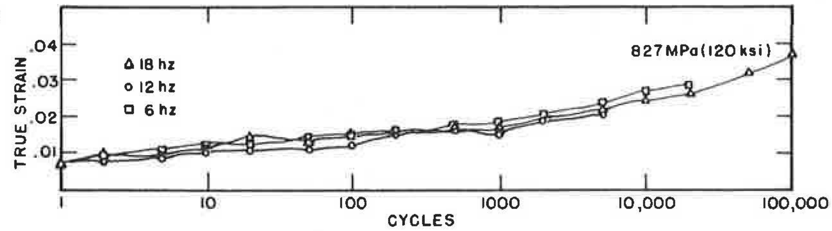
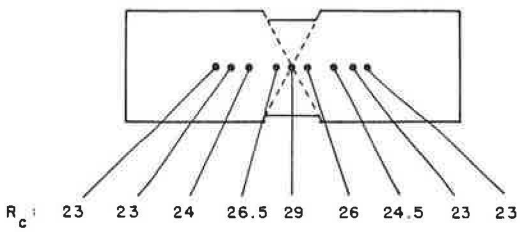


Figure 8. Schematic of hardness traverse (values in Rockwell C hardness numbers) across deformed region of standard rail steel specimen [σ = 1102 MPa (140 000 lbf/in²)].



or if the tests were run in an aggressive environment, a frequency effect could be expected.

Extent of Deformation

Because more than one region of each sample was used for tests, it was desired to determine how much material on either side of a deformation region was affected. A hardness traverse was therefore made across the deformed area of a specimen (see Figure 8). The results showed that the hardening effect was well confined directly under the dies.

RESULTS AND DISCUSSION

Simple Loading

The results for the hot-rolled carbon steel rail are shown in Figure 9. At all stress levels, strain increased as the number of cycles increased, which indicated that the steel was softening. As the stress level increased, so did the rate of softening and the total amount of softening.

It appears that, at 896 MPa (130 000 lbf/in²) or less, the softening behavior would eventually stabilize if the cycling were continued beyond 100 000 cycles. However, above 896 MPa, the metal appeared to soften continuously. Cycling at the higher stresses was not continued to 100 000 because the sample thickness had been reduced 10 percent and cracks were forming. It was presumed that those samples would fail before the softening stabilized.

The other steel types also showed increased softening with increasing stress (see Figures 10 and 11). Note, however, that the effect is less severe for the heat-treated rail and the softening effect is even more damped for the pearlitic CrMo steel.

The increasing deformation resistance is probably partly a function of the increasing hardness and yield strength of the steels. However, the heat-treated carbon steel had slightly greater hardness and yield strength than the pearlitic CrMo steel but exhibited worse deformation behavior. Therefore, other factors must be considered in the deformation behavior. The alloying additions in the CrMo steels might increase deformation resistance by inhibiting dislocation movements.

Split Loading

The results of the split-loading tests are less clear than those for simple loading. In the case of the hot-rolled carbon steel, preloading at a lower stress caused an increase in the softening rate at the higher stress (see Figure 12). For the heat-treated rail (see Figure 13), preloading improved the deformation resistance at 100 and 10 000 cycles and decreased deformation resistance when the load was increased after 1000 cycles. Preloading the pearlitic CrMo steel improved its deformation resistance at the higher stress (see Figure 14).

The reasons for the variable effects of preloading on the different steels are not clear. Preloading of the plain carbon steel may promote dislocation movement in the pearlite, whereas the alloying additions in the other steels may inhibit dislocation movement and thus improve deformation resistance at the higher loads. The heat-treated rail, at 1000 cycles, must reach some critical dislocation arrangement that promotes the increased softening behavior. Further study is under way to explain this behavior more fully.

Microstructure of Deformed Specimens

The microstructure of a deformed specimen 0.5 mm (0.02 in) below the surface is shown to be comparable to that of service-deformed rail steel (see Figure 15).

Prediction of Plastic Flow

Deformation as a function of stress, number of cycles, and microstructure can be calculated by a modified form of an equation developed by Langford (8) for deformation caused by cold rolling. Langford's equation for axisymmetric compression of pearlite is

$$\sigma = \sigma_0 + (k/\sqrt{2d}) \exp(\epsilon_p/2) \quad (5)$$

where

σ = compressive stress,
σ_0 = friction stress (76.4 MPa),
k = Hall Petch constant (0.5 to 0.68 MN/m$^{3/2}$),
d = pearlite spacing, and
ϵ_p = plane strain.

Rearranging terms,

$$\epsilon_p = 2\left\{\ln\left[(\sqrt{2d}/k)(\sigma - \sigma_0)\right]\right\} \quad (6)$$

and, from Equation 4,

$$\epsilon_t = (4/\sqrt{3})\left\{\ln\left[(\sqrt{2d}/k)(\sigma - \sigma_0)\right]\right\} \quad (7)$$

Figures 9-14 show that ϵ_t is made up of the strain after one cycle ϵ_1 and the cyclic strain ϵ_c if more than one cycle is considered. In addition, each increment of ϵ_c is accompanied by the log of a cycle of stress. Therefore, for more than one cycle of stress, Equation 7 can be modified as follows to fit the curves presented:

Figure 9. Cyclic deformation behavior of standard hot-rolled plain carbon rail steel.

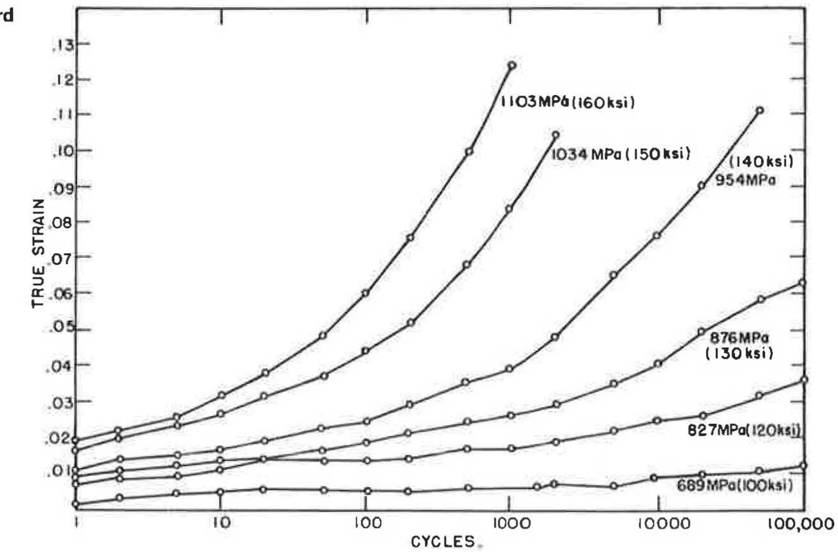


Figure 10. Cyclic deformation behavior of fully heat-treated plain carbon rail steel.

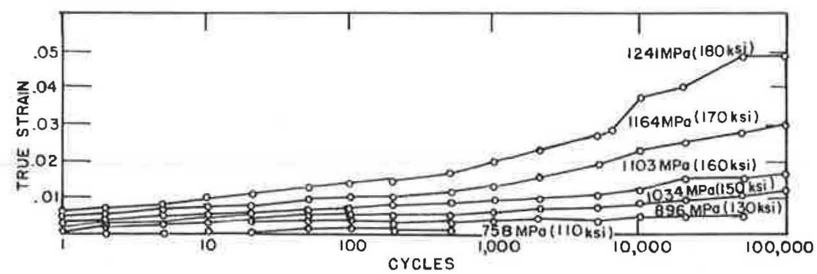


Figure 11. Cyclic deformation behavior of pearlitic CrMo rail steel.

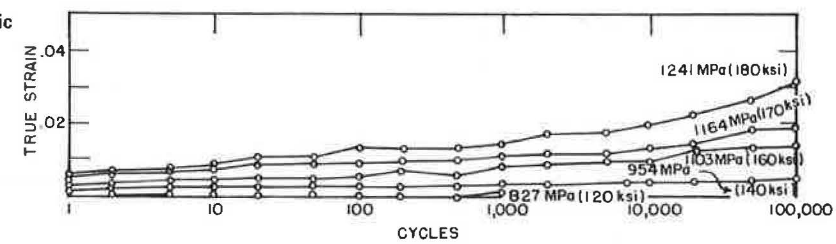


Figure 12. Effect of split loading on deformation resistance of standard carbon rail steel.

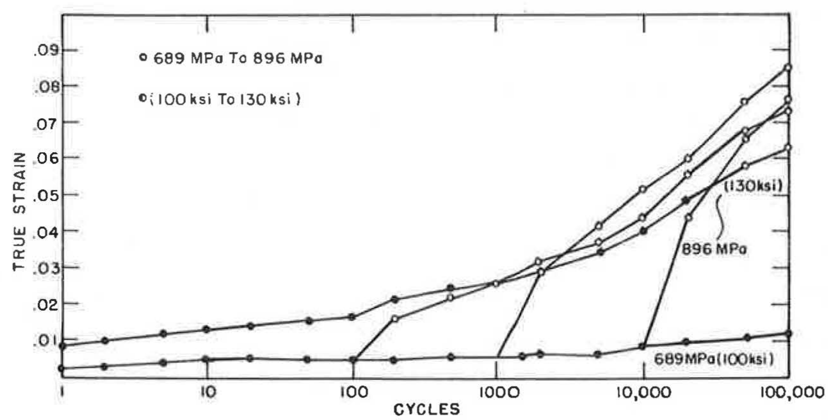


Figure 13. Effect of split loading on deformation resistance of heat-treated carbon rail steel.

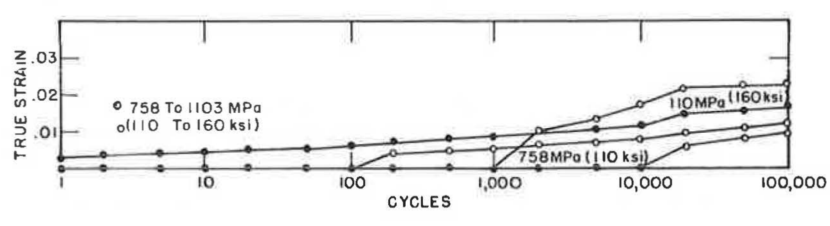


Figure 14. Effect of split loading on deformation resistance of pearlitic CrMo rail steel.

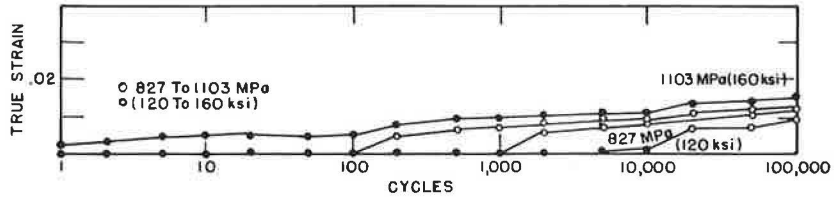
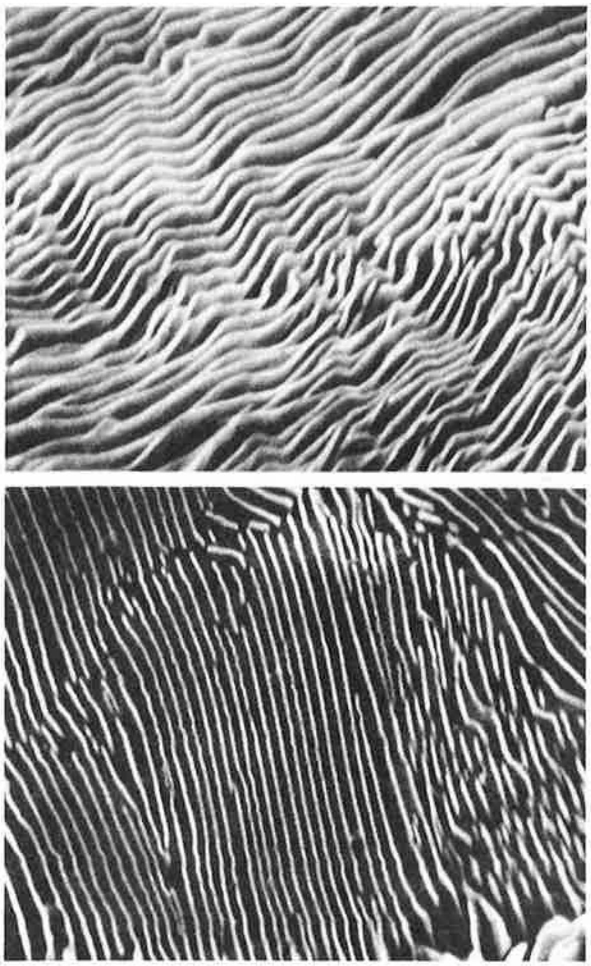


Figure 15. Microstructure of standard rail steel specimen 0.5 mm (0.02 in) below deformed surface (4900X etched in Nital).



$$\epsilon_c + \epsilon_i = (4/\sqrt{3})\, A \ln N \{\ln [(\sqrt{2d}/k)(\sigma - \sigma_0)]\} \quad (8)$$

Evaluation of the curve, for 689 MPa (100 000 lbf/in^2) in Figure 9, gives a mean value of 0.17 MN/m (11 680 lbf/ft) for A, and Equation 8 reduces to

$$\epsilon_c - \epsilon_i = 4.27 \times 10^{-3} \ln N \{\ln [(\sqrt{2d}/k)(\sigma - \sigma_0)]\} \quad (9)$$

A set of experiments was performed by Code (9) in which brass pins were inserted in railheads that were then placed in service. The rails were removed after varying amounts of traffic load and were sectioned, and the deformation of the pins was measured. For two pins, after 68 million gross Mg (75 million gross tons) of traffic, the average true strain can be calculated from Code's data as 0.023 and 0.017. In 1951, the average freight carload was 38 t (42 tons); adding 28 tons for the empty car weight gives a gross load of 63 Mg/car (70 tons/car). This provides an estimate of 4.3 million cycles for the duration of Code's test (number of cars times four axles). Substituting this value into Equation 9 gives a strain of 0.016. This value is in close agreement with the values measured in the field, which run from a maximum of 0.046 at the surface to 0 at a distance of 8 mm (0.34 in) below the surface.

CONCLUSIONS

1. When cycled in plane strain compression, plain carbon steels, heat-treated steels, and CrMo steels exhibit softening. The rate and the amount of softening increase with increasing stress.
2. Deformation resistance increases with increasing hardness and yield strength of the steel.
3. The behavior of steels under split loading varies depending on steel type. The reason for the difference is not clear.
4. The wavy pearlite microstructure developed in rails during service is duplicated in laboratory specimens.
5. The average flow in rails can be predicted if stress, microstructure, and number of cycles are known.

REFERENCES

1. Yearbook of Railroad Facts. Assn. of American Railroads, Washington, DC, 1979.
2. B. N. Leis. Cyclic Inelastic Deformation and Fatigue Resistance Characteristics of Rail Steels. In Rail Steels: Developments, Processing, and Use (D. H. Stone and G. G. Knupp, eds.), ASTM, Philadelphia, Special Tech. Publ. 644, 1978.
3. S. Marich and P. Curcio. Development of High-Strength Alloyed Rail Steels Suitable for Heavy-Duty Application. In Rail Steels: Developments, Processing, and Use (D. H. Stone and G. G. Knupp, eds.), ASTM, Philadelphia, Special Tech. Publ. 644, 1978.
4. Y. J. Park and D. H. Stone. Cyclic Behavior of Class U Wheel Steel. ASME, New York (in preparation).
5. M. A. P. Dewey and G. W. Briers. Structure of Heavily Cold Drawn Eutectoid Steel. Journal of Iron and Steel Institute, Vol. 204, 1966, p. 102.
6. G. Langford. A Study of the Deformation of Patented Steel Wire. Metallurgical Trans., Vol. 1, 1970, p. 465.
7. A. B. Watts and H. Ford. An Experimental Investigation of the Yielding of Strip Between Smooth Dies. Proc., Institute of Mechanical Engineers, Vol. 1B, 1952, pp. 448-453.
8. G. Langford. Deformation of Pearlite. Metallurgical Trans., Vol. 8A, 1977, p. 861.
9. C. J. Code. Determination of Plastic Flow in Rail Head. AREA Bull., Vol. 59, 1958, p. 962.

Publication of this paper sponsored by Committee on Track Structure System Design.

Nondestructive Testing of Railroad Rail

Harold Berger

Techniques of nondestructive testing (NDT) of railroad rail in service are reviewed with the aim of assessing the state of the art and future needs. The contributions to the industry of the primary NDT methods—ultrasonic and magnetic inspection—are noted, and their limitations are examined. The limitations of ultrasonic inspection include ensuring the coupling of the ultrasonic signal into and out of the rail, setting the sensitivity level of the inspection system reproducibly, and relating the amplitude of the return ultrasonic signal to the size of the defect. Magnetic inspection is generally limited to the railhead. The two systems used together provide the most reliable inspection, the magnetic system providing special assistance with defects located near the edges of the railhead. Recommendations for improving rail NDT include greater use of these two complementary systems (now available on only about 50 percent of U.S. rail test cars), greater attention to operator training and characteristics and to the inspection of new rail before installation, and changes in government regulations that will lead to more effective use of rail test cars. In addition, research is needed to relate defect growth to rail service conditions so that realistic decisions can be made about leaving defective rail in use. Developments leading to improved technology are also discussed.

The subject of nondestructive testing (NDT) of railroad rail is a complex one that involves technical considerations (such as types of inspection, the types and sizes of flaws that can be detected, and the reliability of detection), economic considerations (such as how often to inspect and the sizes of defects that should lead to rail replacement), and regulatory questions (such as how much leeway railroads should be permitted in inspection and rail replacement). Obviously, the performance of rail depends strongly on traffic density, axle loading, condition of equipment (flat spots on wheels, for example), and many other factors. This paper focuses on the technical aspects of rail NDT but also attempts to take some of these other issues into account.

Field NDT of railroad rail began in an organized way in the United States in 1928 with the introduction of a rail test car designed by Sperry (1). The NDT method used in this first car was an inductance method in which variations in the electromagnetic field induced by electric current in the rail were sensed by a pickup coil (2, 3). Since that early work, other magnetic and ultrasonic methods have come into prominence in the field inspection of railroad rails (4). In addition, eddy-current, liquid-penetrant, and magnetic-particle methods are used to assess new rails and/or repairs (5-7).

The use of these NDT procedures to inspect in-place rails has undoubtedly been of great benefit to the nation's railroads. In 1965, Magee of the Association of American Railroads (AAR) estimated that rail inspection saved the railroads almost $200 000 000/year by minimizing service failures and the costs associated with these failures (1). Although rail failures have not contributed in a major way to railroad fatalities, the inspection program has also saved lives.

There have been obvious benefits from rail inspection. Yet, in the past few years—as Figure 1 (8, 9) shows—there has been an increase in track-related accidents while other causes of train accidents have stayed about constant. Rail problems remain a significant factor (9). The number of defective rails found by NDT and replaced each year showed a dramatic rise in the late 1960s and early 1970s and now appears to have leveled off somewhat (10), in spite of the fact that total kilometers of track in the United States continues to decrease each year (11). We must also consider the increasing axle loading in recent years as 91-Mg (100-ton) cars have become more common [see Figure 2 (11)], and we must recognize that the increased loading on remaining rails may cause defects to grow at a faster rate. This means being alert to the possible need for detecting smaller rail defects or inspecting rails more frequently.

NONDESTRUCTIVE TESTING METHODS

Ultrasonic Testing

Ultrasonic inspection is normally done in a pulsed mode; some portion of the ultrasonic pulse is reflected or scattered back to a receiving transducer, sometimes the same transducer that transmitted the energy in the first place. The ultrasonic pulse travels at a known velocity in the rail [typically 5900 m/s (19 350 ft/s) for longitudinal waves]. Therefore, the time at which the reflected signal is received can be related to a distance in the metal rail. One can use electronic gates on the ultrasonic instrument to confine the inspection to a certain region, if desired. Typical ultrasonic frequency is 2.25 MHz.

Ultrasonic testing has many advantages for detecting cracks and similar discontinuities. A large part of the ultrasonic signal is reflected from such interfaces. However, if the receiving transducer is to receive a significant amount of the reflected energy, it must be in the correct location.

An easy flaw to detect is an extensive horizontal split head (see defect 1 in Figure 3) because an ultrasonic beam directed straight down the head toward the web and base will be reflected back toward the transducer. This same (0°) transducer also provides good inspection of the web area. Note, however, that, if the defect is a horizontal split head that does not extend over the web area (defect 2 in Figure 3) or a vertical split head (defect 3 in Figure 3), a transducer aimed straight down will probably receive only a very small reflection signal. To detect such defects by using ultrasound, it is necessary to use angled ultrasonic beams. Common angles, from the vertical direction, are 30°-60°, 37.5°, 45°, and 70°-80°. The capabilities of such angled beams for detecting common defects in rails are summarized in a report by Kaiser and others (12).

If the types of rail defects can be anticipated, the proper angular ultrasonic beam and the preferred location of the receiving transducer can be determined. In practical situations, however, there are some problems. For example, the transducers must be coupled to the rail in order to get as much ultrasound energy as necessary into and out of the rail. Two common approaches are to place the transducers in a liquid-filled rubber wheel or in sleds that slide along the rail. Liquid coupling is used between the wheel or sled and the rail. Both of these approaches impose some restrictions on the angular orientations and the number of transducers that can be used. Note that the curvature at the edges of the top of the railhead makes it necessary to place the transducers above a point near the center of the rail. This contributes to the difficulty of detecting defects located near the edges of the railhead. Defects located near bolt holes, rail ends, or welds also present problems because the reflections from these interfaces mask closely located defects.

Another significant problem is that of maintaining coupling while the test car is moving. Obviously, some

Figure 1. Train accidents by major cause (at inflated thresholds).

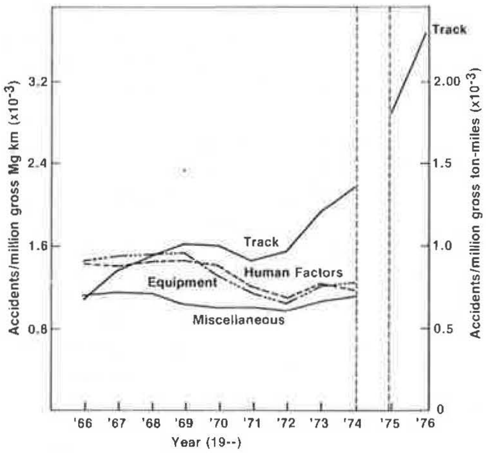


Figure 2. Average freight car capacity on U.S. railroads by year.

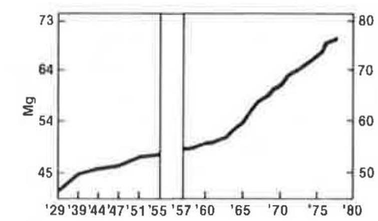
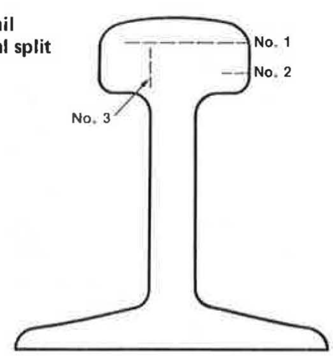


Figure 3. Orientations of several rail defects: defects 1 and 2, horizontal split heads; defect 3, vertical split head.



bounce is introduced. This can be checked for the 0° beam because the equipment should show a strong back reflection from the bottom of the rail base. For some of the angled beams, this is not the case, and coupling can be lost or become intermittent without the operator realizing it. In some cases, however, the angled transducers are deliberately located in the same coupling area as the 0° transducers to provide at least some indication of coupling.

Checking the sensitivity of rail inspection equipment is a significant problem. Ideally, one wants to adjust equipment sensitivity by using the same material and geometry as those of the object to be inspected. Naturally, it is difficult to carry a rail around and get it into position to test. A good sensitivity check would therefore involve something already in the rail. The Southern Railway System, for example, uses the reflection from bolt holes to set sensitivity. This seems to be a reasonable approach, but it does not work well for many of the angled beams. Certainly, the question of sensitivity adjustment merits further consideration.

Among other problems is the fact that surface defects, such as burns and shells, and welds often interfere with the transmission of ultrasound and therefore impede the detection of defects below those areas.

The presentation and interpretation of data are also important, of course. Most U.S. rail-car systems use pen recorders. The electronics system gates the return pulses so that only those pulses of interest cause pen deflection. A typical system described by Thomas and others (13) has two pens tied to a 0° ultrasonic transducer. One pen detects reflections in the head, web, or base of the rail; the other notes a loss of base reflection. In this system there are two ultrasonic transducers at 37.5° and one recorder pen is tied to each transducer. There are also two transducers at 70°, both of which are tied to a single recorder. The operator must watch the recorder pens (five in this case) and watch the track through an operator's window [see Figure 4 (14)]. If a suspicious signal is detected, the test car is stopped and the operator gets out to check the suspect area with a portable ultrasonic system. The hand check indicates either that the signal was a false alarm or that it was valid. In the latter case, some sizing of the defect is attempted.

Inspection speeds for rail test cars in the United States are in the range of 6-21 km/h (4-13 miles/h) (12, 15). Although inspection speeds in some other countries are reported to be as high as 100 km/h (62 miles/h) (12, 15), many foreign rail cars appear to operate in the 32- to 40-km/h (20- to 25-mile/h) range. In many of these cases, the inspection car does not stop to investigate; inspection data are recorded and analyzed later (15). In the United States, where rail cars stop to confirm and size defects, the average inspection speed is about 11 km/h (7 miles/h).

In some cases, a hand-operated ultrasonic unit, pushed along the rail by a walking inspector, is used to check rail (3, 7). These instruments are used to inspect areas that are not tested by the rail cars (switches, for example) and to go over critical areas such as rails associated with tunnels or bridges.

Magnetic Inspection

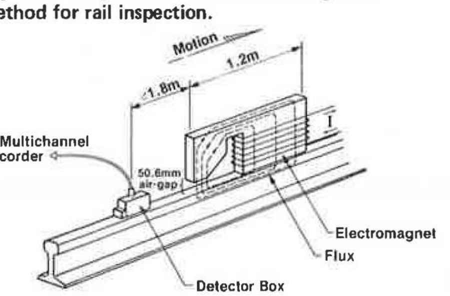
Two basic approaches are used in magnetic testing of rail. One is the inductance method used in the original rail test car by Sperry (1-3). In this method, current is put into the rail either by direct contact or by the movement of a strong magnetic field. Perturbations in the current flow that are caused by defects are detected by pickup coils. In the other approach, a magnetic field is set up in the rail. Flaws perturb the magnetic field, and variations in the residual magnetic field are sensed by a detector several meters away (to avoid detection of the active magnetic field). A diagram of a typical unit is shown in Figure 5 (13). This unit provides a longitudinal magnetic field, which yields good detection of transverse defects that would break the magnetic flux lines. In addition, magnetic fields are usually introduced across the railhead by following this unit with one that has a permanent horseshoe magnet whose pole pieces are at either side of the railhead. The resulting transverse magnetic field provides good detection of longitudinal defects.

Magnetic testing is usually limited to inspection of the railhead. Therefore, the number of rail defects discovered by using magnetic testing is generally less than the number found by using ultrasonics. Nevertheless, the two methods complement each other and are sometimes used together on the same test car.

The speed of magnetic inspection cars falls in the same range as that of ultrasonic cars; the higher speed

Figure 4. Ultrasonic rail test car used by British Railways (note operator in car window at right).

Figure 5. Diagram of AAR residual magnetic method for rail inspection.



Note: 1 m = 3.28 ft; 1 mm = 0.039 in.

of 21 km/h (13 miles/h) represents a magnetic system on good track. Kaiser and others (12) estimate that the speed capabilities of the electric-current or eddy-current methods can be as great as 80 km/h (50 miles/h) whereas the top speed for residual magnetic systems (systems of reasonable size) may be only about 27 km/h (17 miles/h).

Although coupling the magnetic field into the rail sounds simpler than what is required in ultrasonics, bouncing of the magnetic yoke and changes in the small air-gap structure on either end can lead to variations in the magnetic field that introduce false indications.

CURRENT PROBLEMS IN RAIL TESTING

NDT methods for rail provide signals that must be interpreted by the operator. Although a considerable amount of money has been spent on rail-testing equipment, it is less clear how much effort has been expended, on an industrywide basis, to ensure that operators are well trained and motivated. Since the operator is such a vital part of the NDT process, some additional industry effort would seem to be appropriate in this area.

The problem of operator performance is not unique to the rail industry. In fact, the nature of the problem is indicated by this quotation from a representative of an aircraft manufacturer, quoted by Robinson (16): "The burden is on the inspector to react on the spot and make a go or no-go decision on the basis of not altogether unambiguous information."

It is recognized that differences in the inspection equipment used by individual railroads present something of a problem in training operators. It is also recognized that the operator's motivation can outweigh technical training when it comes to the quality of inspection. Therefore, even if industrywide training of NDT operators is not pursued, research into the behavioral and psychological areas should be accelerated. It would be extremely beneficial to know what characteristics a good inspector should possess and how motivation can be maintained.

The NDT equipment currently available for rail testing is basically capable of performing the task, but one has to be aware of its limitations. In using ultrasonic equipment, for example, one depends heavily on the amplitude of a reflected signal. Yet, for several reasons, the amplitude may be low: If the defect is poorly oriented, the ultrasonic coupling is not good, or the calibration of the instrument is off, a low-amplitude signal can result. Since as many as six 70° ultrasonic transducers in a rail car are often tied to one recorder pen, it is entirely possible that one transducer may not be working properly. Yet this would not be known to the operator. Although we may not be able to do anything about poorly oriented defects, we can pay attention to calibration and coupling problems.

Some of the rail defects that grow in service undoubtedly stem from defects that were in the rail as delivered. Greater attention should be given to inspecting new rail to detect inclusions, piping, and similar defects that are likely to grow in service and lead to later rail replacement. Although this would not eliminate the defects that are caused by rail stresses, flat wheels, and other difficult conditions and thus would not eliminate in-service inspection, it would reduce costs for rail replacement in the field. It would probably also raise the average speed of rail test cars because fewer stops might be needed.

About 70 rail test cars are currently in use in the United States. About two-thirds of these are owned and operated by the railroads themselves (3), and the other third are owned and operated by the Sperry Rail Service (10). Except for one ultrasonic car used on the New York subway system, all of the Sperry cars use combined inductance-ultrasonic systems. Of the cars owned by U.S. railroads, about 10 percent are magnetic cars (residual magnetic method) and about 25 percent use combined residual magnetic-ultrasonic systems. The remaining 65 percent are ultrasonic cars.

In recent years, the use of ultrasonic inspection for rails has been emphasized. This is demonstrated by the preponderance of ultrasonic systems among the rail test cars and also by current development efforts in this area.

Nevertheless, the use of two complementary inspection systems is desirable. A combined ultrasonic-magnetic system appears to offer greatly improved defect detection (and signal interpretation) for the railhead. The additional use of the magnetic system offers more reliable detection for common railhead problems, such as vertical split and detail fractures. When these defects are located near the edges of the railhead, magnetic inspection is most useful. For other defects, particularly those located in the web or base of the rail, ultrasonics will continue to provide the best inspection.

It would appear that U.S. railroads are not testing rails as extensively as they could with currently available equipment and personnel. As indicated before, test cars typically provide an average speed of 11 km/h (7 miles/h). By taking 70 available cars and allowing each car to operate 8 h/day at 11 km/h, the industry could potentially inspect more than 6000 km (3600 miles) of track a day (by using only one shift). This translates into a capability to inspect almost 1.5 million km (0.9 million miles) of track a year. Certainly this would be reduced by factors such as track availability. Nevertheless, the actual number of kilometers of track now tested is probably considerably less than half that figure (10).

Although there may be some discussion about the numbers, there seems to be agreement about the conclusion that greater use could be made of available rail test cars. Part of the problem is certainly government regulation, in that railroads are compelled to remove defective rail quickly (often within 24 h) or drastically reduce train speeds. What often happens is that the rail test car is sent packing once it has located an amount of rail that can be replaced by the maintenance crews. A better approach would be to determine likely rates of flaw growth based on service conditions and to take this into account in the replacement schedule. It will require more extensive work, however, to relate flaw growth to anticipated stresses.

CURRENT DEVELOPMENTS

A few words about developments in progress in rail NDT may be appropriate to show the directions in which the industry is headed. Work has been going on for many years on electromagnetic transducers for ultrasound. A major advantage of these transducers is that physical contact with the object under inspection is not necessary and thus coupling problems are minimized. A great deal of work in this area has been reported in the Soviet Union, including potential application to problems of rail inspection (17). Development of electromagnetic transducers is now proceeding in the United States (18), and a test of these novel transducers for rail inspection is planned. In addition to ease of coupling, these transducers offer the potential for electronic variation of the angle of the ultrasonic beam.

The major developments directly associated with rail NDT are in the area of data processing and presentation. Investigators at the Transportation Systems Center (TSC) are working on a consolidated B-scan presentation that would present a cross-sectional view of the rail to the operator when a defect signal is detected and pinpoint the location of the defect (19). Other programs that are being pursued at TSC include an improved magnetic system to complement ultrasonic inspection and a fully automated rail test car that will have the capability for sizing defects. The British are also working on improved data processing. Their present system collects inspection data during night runs of the track. These data are later analyzed in a central location (15). New efforts in Great Britain are directed toward a completely automated system (20).

These appear to be representative of the major development efforts directly concerned with the detection of defects in rail by nondestructive evaluation. Other major problems, of course, are residual stresses and longitudinal forces in rail. These problems have particularly come into focus with the introduction of all-welded rail (21). This paper has not addressed that problem nor problems that are associated with nondestructive evaluation of welds.

Other developmental efforts that will possibly affect rail NDT involve ultrasonic testing and attempts to use portions of the signal in addition to, or instead of, ultrasonic amplitude. As indicated earlier, the amplitude of reflected or scattered ultrasound can be misleading. Current development work includes efforts to use spectral, phase, and other signal parameters to identify the type and approximate size of defects (22-24). These efforts should be followed.

CONCLUSIONS AND RECOMMENDATIONS

Rail NDT is strongly operator dependent, and the equipment has some limitations. Improvements in rail NDT could be made. The following are some near-term recommendations:

1. Pay more attention to the inspection of new rail; elimination of defects at that point can minimize more expensive problems in the field. However, it should be clear that in-service inspection would still be necessary.
2. Consider an industrywide training program for NDT operators. Not much attention appears to have been paid to the operator, a vital part of the present NDT system. At the very least, the railroads should join with other industries to help in determining the characteristics of a good inspector. Strong motivation is probably more important than textbook knowledge.
3. Take steps to be sure present equipment is functioning and the sensitivity is properly set. Adopt widespread use of cars that make use of two complementary NDT methods (ultrasonic and magnetic).

On a longer-term basis, the industry should move toward the following:

1. Work to help set reasonable government regulations and policies so that the full range of NDT capability can be used with minimum penalty. To do this realistically, more work will have to be done to relate flaw growth to rail service conditions.
2. Improve the presentation of NDT data to ease interpretation and minimize errors. On a longer-term basis, the industry should work for the development of more automated test systems. This is necessary if a significant increase in the speed of rail inspection is desired.
3. Develop improved techniques, such as ultrasonic methods in which coupling can be checked or the effects of coupling variations are minimized. Improved magnetic approaches, such as rapidly changing field directions, should also be factored into developmental planning.

These recommendations are made in an attempt to improve rail inspection and reduce the cost (both in terms of money and lives) of rail-related accidents. It is obvious that rail NDT has served the industry well in the past 50 years and will continue to do so in the future. But its impact can be enhanced if improvements are made along the lines suggested here.

ACKNOWLEDGMENT

I am pleased to acknowledge helpful discussion concerning the subject of this paper with many people in the railroad industry. The information presented in this paper was gathered during my service as a member of the Materials Advisory Committee, Research and Test Department, AAR. The committee is chaired by Morris Cohen of the Massachusetts Institute of Technology. Helpful background information was obtained at a one-day Seminar on Railroad Nondestructive Evaluation, held under the auspices of AAR and cochaired by Daniel Stone of AAR. Discussions with Aviva Harvey and George Way of AAR helped me to formulate some ideas relating to railroad accidents and rail problems. In addition, W. J. Harris, Jr., and A. G. Payne of AAR; H. Ceccon, C. Fraser, and A. L. Lavery of TSC; and T. C. Rohr and C. N. Scott of the Southern Railway System provided constructive criticism of the initial draft of this paper. I am pleased to acknowledge their assistance.

REFERENCES

1. G. M. Magee. Preventing Rail Failures in Track. Materials Evaluation, Vol. 23, No. 10, 1965, pp. 508-511.
2. Rail Defect Manual. Sperry Rail Service, Automation Industries, Danbury, CT, 1968, pp. 26-29.
3. The Track Cyclopedia, Section 9. Simmons-Boardman Publishing Corp., Omaha, 1978.
4. W. E. Chapin, R. Kins, H. C. Festel, and R. H. Breslin. A Bibliography of Rail Technology. Federal Railroad Administration, U.S. Department of Transportation, Rept. FRA/ORD-77/15, 1977.
5. J. Banks and D. C. Hansford. Examination of Surface Cracks in Carbon and Manganese Steel Rails by the Eddy-Current Technique. British Journal of Nondestructive Testing, Vol. 18, No. 6, 1976, pp. 174-179.
6. J. Banks. Magnetic Particle and Dye Penetrant Techniques as Aids to the NDT of Rails. British Journal of Nondestructive Testing, Vol. 15, No. 6, 1973, pp. 176-178.
7. A. K. Gurvich and others. Flaw Detection in Rails. Federal Railroad Administration, U.S. Department of Transportation, Rept. FRA/ORD-77/10, Dec. 1978.
8. W. H. Dempsey. Supplement to Statement on Railroad Safety to Subcommittee on Transportation and Commerce of the Committee on Interstate and Foreign Commerce, March 15, 1978. Assn. of American Railroads, Washington, DC.
9. A. E. Shulman and C. E. Taylor. Analysis of Nine Years of Railroad Accident Data, 1966-1974. Assn. of American Railroads, Washington, DC, Rept. R-223, April 1976.
10. AI Sperry Railer. Sperry Rail Service, Automation Industries, Danbury, CT, Vol. 47, No. 1, 1979.
11. Yearbook of Railroad Facts. Assn. of American Railroads, Washington, DC, 1979.
12. W. P. Kaiser, R. H. Byers, D. Ensiminger, H. C. Meacham, J. H. Flora, W. C. Bruce, L. Becker, and G. Posakony. Rail Inspection Systems Analysis and Technology Survey. Federal Railroad Administration, U.S. Department of Transportation, Rept. FRA/ORD-77/39, Sept. 1977.
13. J. M. Thomas, C. S. Davis, T. W. Rettig, and P. R. Bosinoff. Strategy and Methods for Cost-Risk Analysis of Defective Rail. Failure Analysis Associates, Palo Alto, CA, Rept. FAA-78-12-1, Dec. 1978.
14. Ultrasonic Eyes Aboard British Testing Train. Materials Evaluation, Vol. 29, No. 6, 1971, pp. 16A-18A.
15. W. D. Kaiser and R. H. Byers. Rail Flaw Inspection Technology. Paper Summaries, National Fall Conference of American Society for Nondestructive Testing, Denver, Oct. 1978, pp. 393-396.
16. A. L. Robinson. New Testing Methods Could Boost Air Safety. Science, Vol. 205, July 6, 1979, pp. 29-32.
17. V. V. Vlasov, V. A. Lanchak, N. A. Oluhkov, and I. V. Ivanov. Ultrasonic Method of Testing Railway Rail Tracks Using Electromagnetic-Acoustic Converters. Defectoskopiya, Vol. 3, 1971, pp. 94-98.
18. R. B. Thompson. New Electromagnetic Transducer Applications. Air Force Materials Laboratory, Wright-Patterson Air Force Base, Dayton, OH, Rept. AFML-TR-78-55, May 1978, pp. 340-344.
19. D. Lavery and H. Ceccon. State of the Art of Rail NDT in the U.S. Seminar on Railroad Nondestructive Evaluation, National Bureau of Standards, Washington, DC, May 15, 1979.
20. H. Jenkins. Track Maintenance for High-Speed Trains. AREA Bull., Proc. Vol. 77, No. 658, June-July 1976, pp. 511-513.
21. J. J. Yavelak and C. N. Scott. Requirements of Residual Stress Measurements Peculiar to the Railway-Freight Industry. Extended Summaries, Meeting of Society for Experimental Stress Analysis, Indianapolis, Oct. 23-25, 1978, pp. 17-18.
22. B. R. Tittman and R. K. Elsley. Experimental Measurements and Interpretation of Ultrasonic Scattering by Flaws. Air Force Materials Laboratory, Wright-Patterson Air Force Base, Dayton, OH, Rept. AFML-TR-78-55, May 1978, pp. 26-36.
23. L. Adler, K. Lewis, P. Szilas, and D. Fitting. Identification of Flaws from Scattered Ultrasonic Fields as Measured at a Planar Surface. Air Force Materials Laboratory, Wright-Patterson Air Force Base, Dayton, OH, Rept. AFML-TR-78-55, May 1978, pp. 36-43.
24. R. Shankar and A. N. Mucciardi. Application of Adaptive Learning Networks to Ultrasonic Signal Processing: Detecting Cracks in Stainless Steel Pipe Welds. In Ultrasonic Materials Characterization (H. Berger and M. Linzer, eds.), American Society for Nondestructive Testing, Columbus, OH (in preparation).

Publication of this paper sponsored by Committee on Track Structure System Design.

Review of Rail Research on British Rail

C. O. Frederick and E. G. Jones

The rail research program of British Railways, which is aimed at understanding and reducing the severity of various mechanisms of rail failure, is described. An important part of the work is the measurement and prediction of rail stresses and the study of force-free temperature for continuous welded rail. To reduce failure problems, it is necessary to develop laboratory-based techniques to assess the performance of rail steels and welds. This requires a knowledge of the dynamic and static stress environment of the rails and computer programs to calculate these stresses. The study of failures includes the study of Thermit and flash-butt weld failures, tache ovale defects, star cracks at bolt holes, and squat defects. It has been found that the majority of Thermit weld failures can be attributed to poor welding practice. Flash-butt weld failures are much less frequent but may become more of a problem as the more wear-resistant rail steels are introduced into welded track. The need to develop better steels for switch and crossing work has provided an impetus to develop a weldable austenitic manganese steel and also bainitic steels of high strength and toughness. These developments are reviewed.

Rail research should reflect the future objectives and problems of the railways concerned. British Railways (BR) is a high-speed railway, and this has tended to cause rather special problems. In addition, the move to continuously welded rail (CWR), while solving some problems, has created new ones—all of which has provided the impetus for research.

The thermal stresses in long, welded rails are, of course, invisible and are commonly overlooked; nevertheless, they are large and very significant in controlling the modes of failure. In designing a railway with CWR, it is always necessary to choose the force-free temperature with care. Too high a force-free temperature will assist the rapid growth of transverse fatigue cracks and brittle fracture of the rail in winter, whereas too low a force-free temperature will create a buckling problem in the hot weather of summer. After a spate of buckling incidents in 1969, BR raised the force-free rail temperature by 5.5°C to improve track stability. Subsequently, an investigation was conducted into the factors that affect track buckling with a view toward improving track design. The rise in force-free rail temperature may explain why there has been an increase in transverse rail defects in recent years. There have, however, been other problems, two of which are closely associated with high-speed lines: squats and short pitch corrugations.

The current maximum strict axle load on BR is 250 kN. This is higher than on most European railways and lower than U.S. values, which can exceed 298 kN. Rail-crushing and side-wear problems are much less severe in the United Kingdom than in the United States. Nevertheless, rail side wear still limits rail life on curves. For large-radius, high-speed curves, where use of the correct transverse rail profile is important to good riding, it has been necessary to prohibit the practice of transposing side-worn rails. Thus, there are incentives to use more wear-resistant rail. The standard BR rail steel [710-MPa ultimate tensile strength (UTS)] is somewhat less wear resistant than the standard U.S. compositions. However, BR is using an increasing quantity of wear-resistant Union Internationale des Chemins de Fer (UIC) grades (880-MPa UTS) and 1 percent chromium rails (1080-MPa UTS) in sharply curved situations. These rail steels bring with them some welding problems and slightly less toughness. There is a need for simultaneous improvements in wear resistance, toughness, and weldability.

The BR rail research program has sought to establish an understanding of and to lessen the existing problems and also to look to the future to see what might be achieved by new rail steels. This paper is an account of some of the main lines of investigation.

FORCE-FREE RAIL TEMPERATURE

Track Buckling

The force-free rail temperature is defined as the rail temperature at which long welded rail experiences zero resultant longitudinal force. This temperature is determined by the procedures used to install the welded rail. In deciding this temperature, it is desirable to have an understanding of track-buckling behavior. Some early experiments in track buckling were done in Mousehole Tunnel between 1956 and 1959 (1). At first, the buckling theories could only account for the behavior of an infinite sinusoidal irregularity in straight track (2). More recently, a more advanced theory has been developed that allows for an individual irregularity on straight and curved track (3). This is an important advance because it shows for the first time the importance of the longitudinal restraint between the rails and the sleepers. If this restraint is large, it helps to prevent feeding of rail compression into the buckling zone as a misalignment develops. The new theory also shows that there is a fairly clearly defined value of the maximum rise in rail temperature at which thermal buckling will not occur despite the presence of misalignments. This "safe" maximum increase in rail temperature is shown in Figure 1, where it can be seen that, in the curve for temperature increase versus deflection, there is first a peak followed by a trough. The height of the peak is very sensitive to misalignments, but the height of the trough is insensitive to these and provides a better design limit.

The calculated variation in the safe temperature increase with lateral and longitudinal resistance for standard BR track components is shown in Figure 2. Buckling experiments have recently been under way at Old Dalby to check these predictions. The value at which buckling took place was always above the calculated safe temperature increase and was sensitive to rail straightness. After the first experiment, the rail developed a permanent lateral set. Even when the track and rails were laid apparently straight, the built-in set in the rail strongly influenced the buckling behavior.

Rolling Out

There have been many observations of BR in-service rails experiencing a general drop in force-free rail temperature, a phenomenon referred to as "rolling out". This should be distinguished from changes in the distribution of force-free temperature along the track, which is caused by creep of the rail along the track associated with movement through the fastenings or movement of the sleepers in the ballast. The rolling-out effect is caused by the rails becoming longer. The magnitude of this effect has been measured by taking out 240-m lengths of rail, measuring their length, and then replacing them. Reductions in force-free temperature of approximately 6°C in a year have been measured for new rails. It is thought that this effect will stabilize and considerably lessen over the years. Nevertheless, it means that a higher initial rail tension is required to prevent track buckling (1°C is equivalent to 1.7 Mg for 56-kg/m rail).

RAIL STRESSES

Wheel-Rail Contact

It is fitting to start an investigation of rail behavior by considering the stress environment of the rail steel. Clearly, the highest stresses come from wheel-rail contact force. These stresses, however, are very dependent on the precise profiles of wheel and rail. Iterative computer programs have been written to calculate the shape of the contact area for different wheel-rail-profile contact arrangements. Contact at the gauge corner will tend to produce an elliptical contact patch that is long and thin, whereas contact of worn wheel and rail near the rail center line tends to produce a wide and short contact patch. These programs must be iterative, since the dimensions of the contact patch depend on the out-of-plane surface deflections. It is customary to assume elastic material behavior and to assume that the surface deflections are those of a semi-infinite half-plane. The latter assumption is somewhat dubious for gauge-corner contact. Since it is well known that rails plastically deform, it may be thought that the assumption of elastic behavior is also dubious. However, the plastic deformation of rails is something that occurs slowly

Figure 1. Typical buckling response.

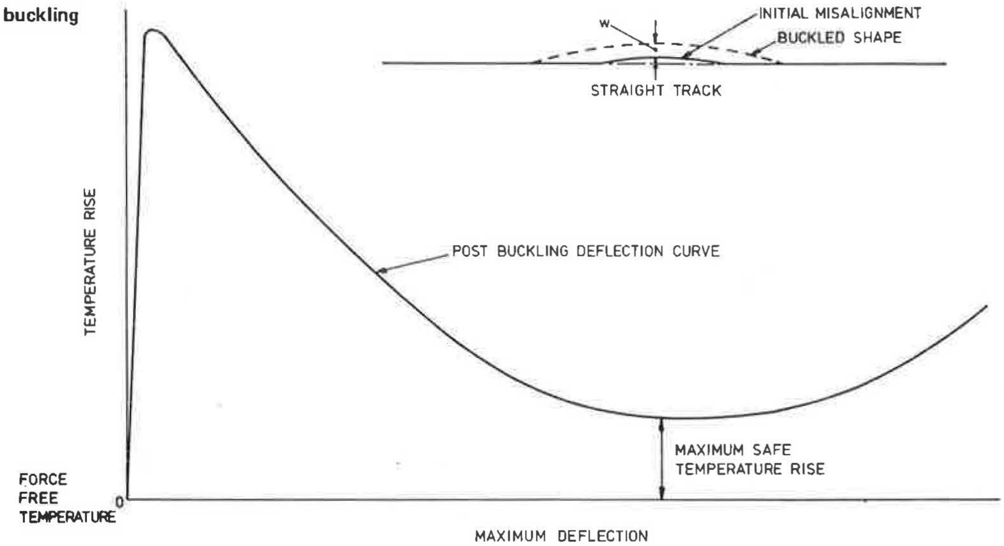
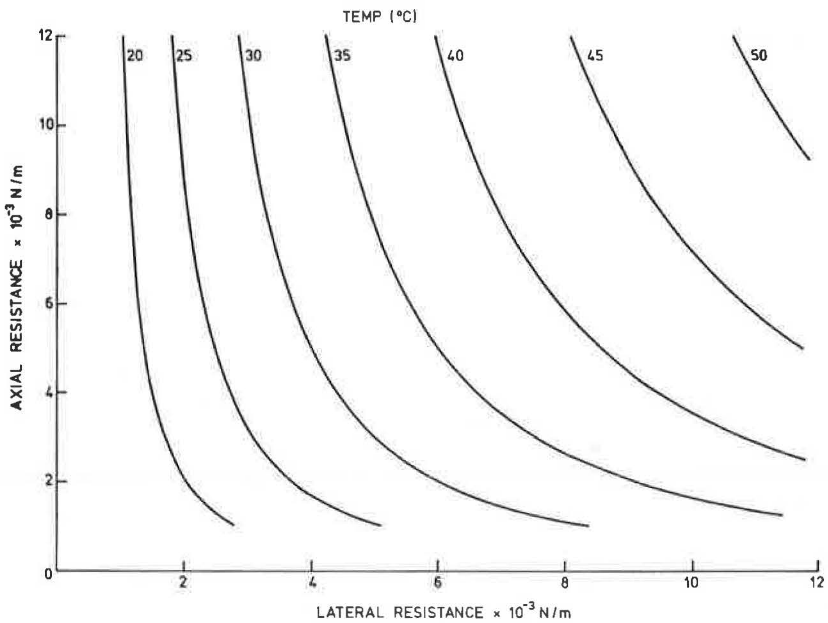


Figure 2. Maximum safe temperature rise for standard BR components.



under many thousands of cycles, and the plastic deformation that occurs under one cycle is almost certainly negligible compared with the elastic strains.

Calculations of the Hertzian stresses caused by wheel-rail contact do not usually allow for flexure of the rail as a beam, nor do they allow for phenomena such as bending of the railhead as a beam supported by the rail web. To investigate these effects, it is usually necessary to resort to finite-element analyses of some sort. These analyses allow the true shape of the rail to be included but make crude assumptions for stresses and strains that occur in the immediate contact zone. When conventional three-dimensional finite elements are used, the computation rapidly becomes very large because of the large number of unknowns. The Research and Development Division of the British Railways Board has found the most promising analysis method to be one that combines finite elements and Fourier techniques (4). This analysis divides the rail up into longitudinal prisms of quadrilateral cross section. Although the loading must itself be expressed in Fourier harmonics along the rail, a good representation of a localized load is possible. This program has been used successfully to analyze the effect of off-center vertical loads on strains in the rail web (see Figure 3).

In the past, lateral forces have been measured for experimental purposes by measuring the bending of the rail web at two positions that are vertically one above the other. This arrangement is, however, affected by off-center vertical loads (5), and this can be demonstrated by using the computer program. In the future, it should be possible to design improved load-measuring systems by using an array of gauges and a small on-line computer.

Dynamic Load Variations

In calculating stresses in rails, it is usually necessary to consider dynamic wheel loads. Dynamic variations in load are especially important at high speeds or when the wheels have formed flats. There are very few data on wheel and rail roughness, but it is clear from calcu-

Figure 3. Calculated rail stresses for off-center vertical load: vertical stresses on the rail web.

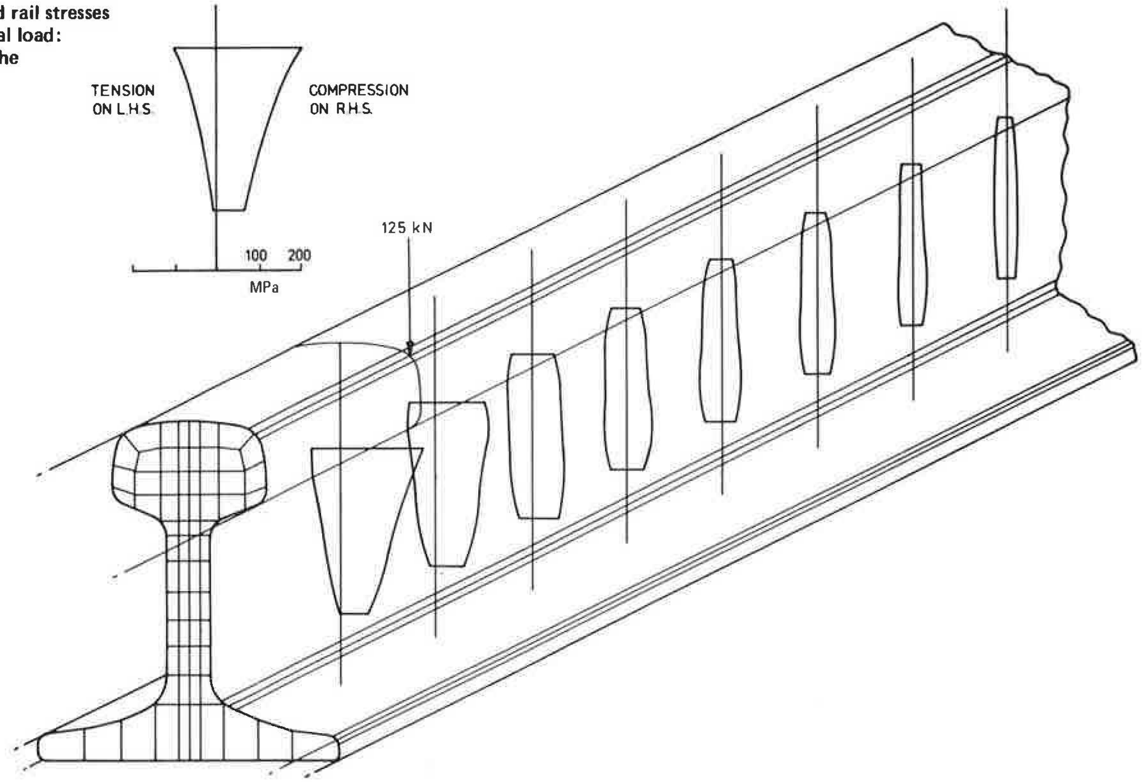
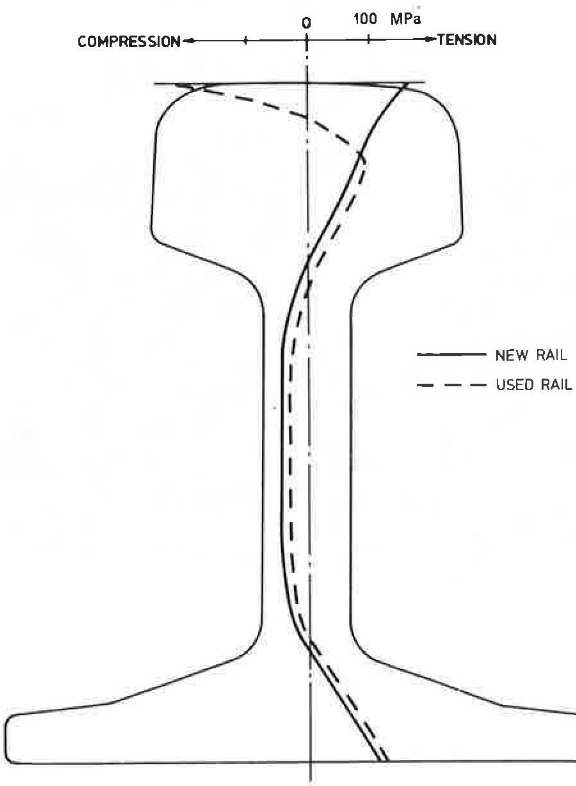


Figure 4. Longitudinal residual stresses in new and used rail on rail centerline.



lations that contact forces can vary substantially with very small levels of roughness (6). These variations are so large because the wheel-rail contact spring is so stiff and because the rail inertia is significant at high frequencies of oscillation. When longitudinal profiles have been measured by using an inertial trolley, it has been found that rail roughness usually decreases under the influence of traffic unless some phenomenon such as corrugation is at work. It is worth noting that, at high speeds, wheel-rail contact forces can be expected to increase if rail weight is increased; thus, mechanisms of rail surface damage are likely to get worse with heavier rails.

Tensile Stresses in Railhead

In considering rail fracture, it is especially important to consider the extreme values of tensile stress in the railhead, since failures rarely initiate in the rail foot (the exceptions in the United Kingdom being defective welds or cracking from corrosion pitting or chair gall). The highest tensile stresses are attributable to a combination of the effects of cold-weather contraction and wheel-flat impact. Thus, the tensile stresses caused by hogging of the rail at either side of the point of impact are more important than the stresses immediately under the point of impact. There has been a substantial program of research to study wheel-flat impact (7), and it has been found that the resultant hogging bending waves propagate very rapidly along the rail and lessen only slowly with distance. As a result, all of the rail will experience to some extent the hogging stresses caused by a wheel flat, although the maximum stresses occur where the quasistatic procession wave is combined with the dynamic effect.

The dynamic testing of rails reflects the importance of tensile stresses in the railhead and is described later in this paper.

Residual Stresses

Recently, there has been an increased interest in residual stresses among BR researchers. It is well known that European rails show high residual tensile stresses

Figure 5. Computer algorithm for correction of an irregular weld.

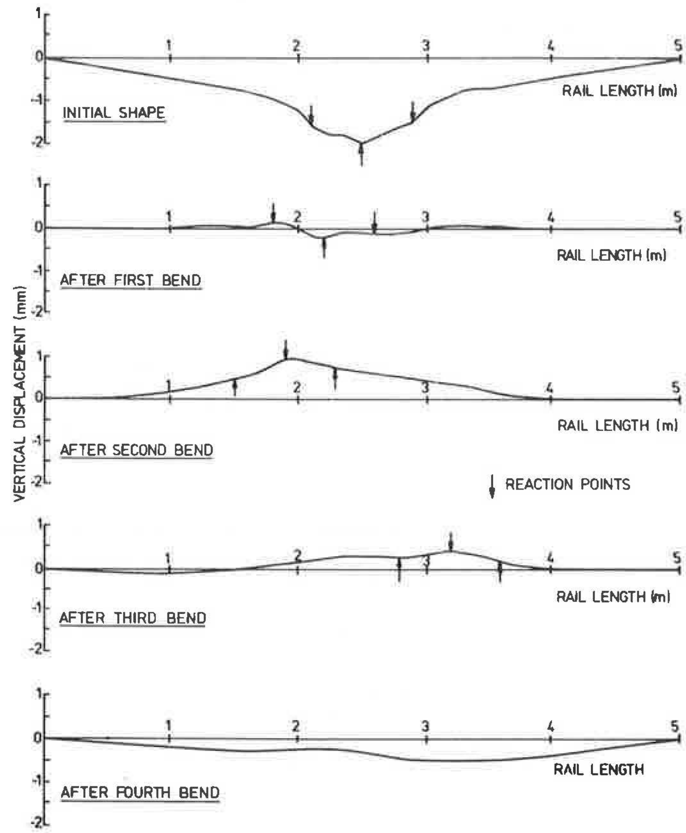


Table 1. Physical properties of rail steels.

Material	Coefficient of Linear Expansion ($^\circ C^{-1} \times 10^6$)	Young's Modulus (GPa)		
		-15°C	20°C	50°C
BS 11 normal quality	11.25	216	213	211
UIC grade B	11.4	211	208	206
1 percent chromium	11.8	216	213	212
Cast LCAMS	17.0	181	180	178
Rolled LCAMS	17.1	201	200	197
Bainitic[a]	11.6	211	209	207

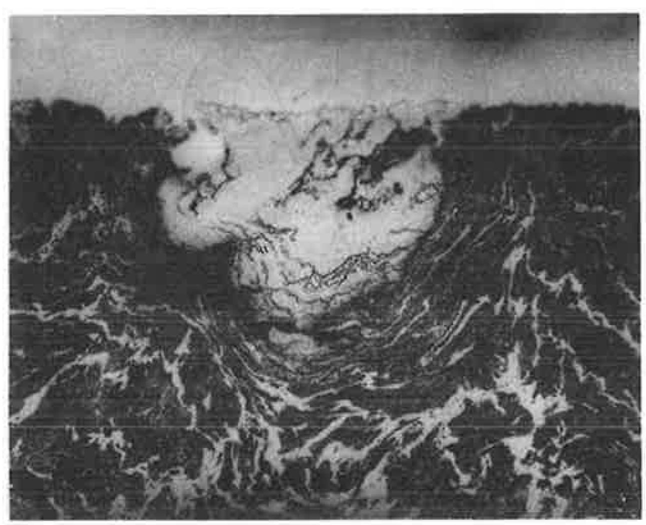
[a]Composition (percentage by weight) = 0.16 carbon, 0.19 silicon, 1.01 manganese, 0.23 sulfur, 0.025 phosphorus, 1.40 chromium, and 0.53 molybdenum.

in the railhead after manufacture and that these stresses are modified by traffic loads (8) so that the longitudinal residual stresses near the running surface are converted from tensile to compressive stresses (see Figure 4). According to German researchers (9), the original residual tensile stresses are caused by the roller straightening process. These residual stresses in the railhead are undesirable because (a) they must increase the magnitude of the rolling-out process and (b) they will probably increase the tendency of cracks to propagate downward across the rail section rather than parallel to the rail surface.

Alternative systems of rail straightening are being examined, and the possibility of a computer-controlled gag press is being considered. Such a device would require a preliminary measurement of longitudinal rail profiles (vertical or lateral) followed by a traverse of one or more automatically controlled presses that obey pressing instructions worked out according to a computer algorithm. Such a system would not produce large residual tensile stresses in the head of the rail.

Work is proceeding to identify suitable computer algorithms. Figure 5 shows the effect of one computer algorithm in correcting an irregular vertical weld profile.

Figure 6. White phase embedded in rail running surface.



Physical Properties

Thermal stresses in CWR are, of course, dependent on the temperature change from the force-free condition, but they are also dependent on the product of Young's modulus and the coefficient of expansion. There are slight differences (±5 percent) in the values used in different countries for the coefficients of expansion of pearlitic rail steels. These differences could be very significant when it comes to selecting a rail steel. We decided, therefore, to obtain some reliable comparative data. The results, which are given in Table 1, seem to show that any variations between pearlitic and bainitic steels are unimportant, although these two steels are very different from austenitic steels.

LIMITS ON RAIL LIFE

Sleeper Condition

The bulk of rails withdrawn from BR service are withdrawn as a consequence of the normal track relaying pattern. The commonest determinant of relaying priority is poor sleeper condition, since there are still many lines with timber sleepers. It has been found that rails removed from straight or slightly curved track for this reason are no longer suitable for use in high-speed lines because of rail-end batter, corrosion, galling of the rail seating area, or localized loss of railhead profile. Nevertheless, the 710-MPa UTS rail steel in current use performs adequately with long-lived sleepers and gives a life of more than 20 years in most lines. More sharply curved track, however, presents a very different picture, and the life of 710-MPa UTS rail can be much less than 2 years. This has led to some use of 900-MPa UTS rail steels. The use of these steels is currently restricted by welding problems and to some extent by availability. Chromium rail steel with 1080-MPa UTS has so far been used on a very restricted experimental basis.

Corrugation

Corrugatory wear on both straight and curved track leads to a shortening of service life and, although its appearance has been reported since the 19th century, the causes are still not understood. The incidence of this phenomenon on BR is increasing, and at present the only remedy is periodic grinding, which obviously will lead to a shortening of rail life. The effect of corrugations on general track deterioration is slowly emerging and is giving cause for concern.

The loosening of iron shoulders cast into concrete sleepers has been observed where rails are severely corrugated, and it is also thought that the corrugations may shorten the fatigue life of the rail. The steel at the crests of corrugations is often severely deformed plastically, and small cracks have been observed that follow the same direction as the plastically deformed grain boundaries. In a joint exercise with Speno International SA to study the effectiveness of rail grinding, it was found that ground rails subsequently developed squat defects, which may well have initiated from cracks that were not completely removed in grinding (10).

Corrugations on high-speed lines usually exhibit patches of "white phase" on the crests. This is very similar in appearance to martensite and is very hard. The possibility that corrugations worsen because of differential rates of wear or corrosion is currently being examined. In this context, the presence of white phase could be important, since it is frequently observed on BR rails. White phase could also play a role in surface-initiated fatigue mechanisms since, as Figure 6 shows, hard pieces of white phase can become embedded in the surface and cause severe plastic deformation (11). These investigations are still exploratory, and the importance of white phase in surface damage mechanisms has still to be ascertained.

Rail Fracture

Sometimes rail fracture causes premature withdrawal of rails from service. In a passenger-carrying system, safety is of the greatest concern, and so some forms of fracture are regarded as more serious than others. The most dangerous forms are those in which a piece of the running surface is removed. In general, a single transverse fracture in CWR is not so dangerous and is usually detected promptly by its effect on track circuits.

In plain track, the following fracture types are of prime interest:

1. Squat fractures—surface defects initiated by rolling-contact fatigue that propagate at a shallow angle and then turn down to form transverse fractures (see Figure 7);
2. Tache ovales—in the United Kingdom, hydrogen-flake-initiated fatigue fractures in the center of the railhead;
3. Star cracks—fatigue-initiated cracks that start in the bore of fish bolt holes;
4. Wheel burns—isolated depressions in the running surface of the rail that lead to (a) high dynamic stresses and subsequent fatigue cracking or (b) continuous transformation of the running surface of the rail, causing hardened microstructures with subsequent fatigue or brittle fracture; and
5. Weld fractures—Thermit weld failures, which generally initiate from a lack-of-fusion defect, and flash-butt failures, which initiate from "flat spots" (entrapped oxide plates on the weld center line).

In all of these fracture types, final fracture is always by brittle cleavage and causes either a complete transverse fracture through the rail or detachment of a portion of the railhead. A star-crack fracture often removes part of the running surface. Tache ovales and wheel burns are particularly dangerous when they initiate at multiple sites at short intervals along the rail.

TESTING OF RAIL STEELS

As demonstrated above, the service life of rail is reduced by wear, rolling-contact fatigue, fatigue, and brittle fracture. To improve service performance, it first becomes necessary to understand how these mechanisms are induced and how rail steels respond.

The assessment techniques used by BR in the evaluation of rail steels have been described in detail elsewhere (12). The basic approach adopted has been to quantify the environment that the rail is subjected to and then use the data so obtained to define a suitable laboratory test. This allows quicker and cheaper evaluation of possible improved materials. This approach also lends itself to gaining an understanding of how the metallurgical structures of rail steels are affected by the various detrimental environmental mechanisms and leads to a materials design concept.

Laboratory Assessment of Fatigue Life

The occurrence of the squat type of defect in BR track has required the development of laboratory assessment methods for failure under rolling-contact fatigue. Preliminary work to date has been carried out on Amsler twin-disc-type machines and small-diameter specimens. Initial experiments indicated that a liquid contaminant (water in this case) was necessary to induce failures. The work to date has been concerned with studying the effect of different creepages—i.e., the percentage of sliding between the two rollers—and contact stresses on the failure rates of a range of rail steels.

In the Amsler tests, it is customary to fix the load and creepage γ, thereby generating in the test certain levels of contact stress σ_c and traction force T. For normal-grade BR rail steel (710 MPa), it was found that cycles to failure N depended approximately on the square of the contact stress. When the creep rate was varied with a range of steels, it was found that the shortest lives occurred at a creep rate of 0.3 percent for all steels. It is thought that this minimum is associated with zero traction at zero creep and, when creep is plotted against the parameter NT for a given contact stress, the minimum disappears. Figure 8 shows a plot of $NT\sigma_c^2$ versus creep for a range of steels. These results tend to indicate a relationship of the following form:

$$N = (1/T\sigma_c^2) f(\gamma) \quad (1)$$

where the function f depends on the steel. Since the work is at an early stage, this result can only be viewed as provisional; it may be a function of the test machine and the limited variation in possible specimen geometries. Specimens of narrow width deform plastically at the loads used so that the original geometry and contact stress are lost. Work is continuing to determine the effect of geometry on contact stress and may lead to a requirement for a larger-scale test rig.

Figure 7. Longitudinal vertical section through a typical squat defect.

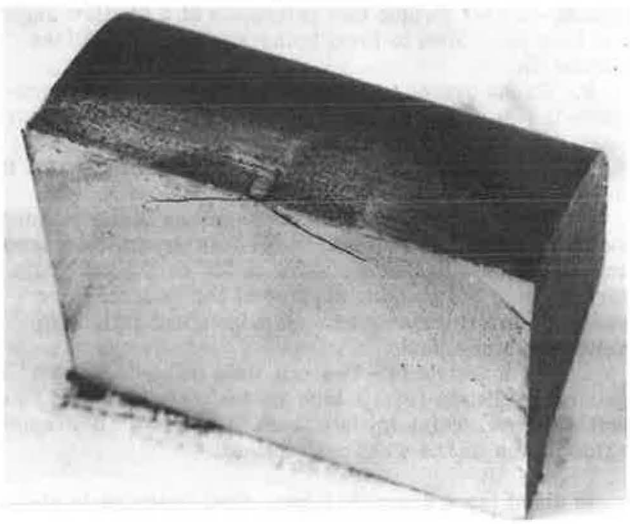
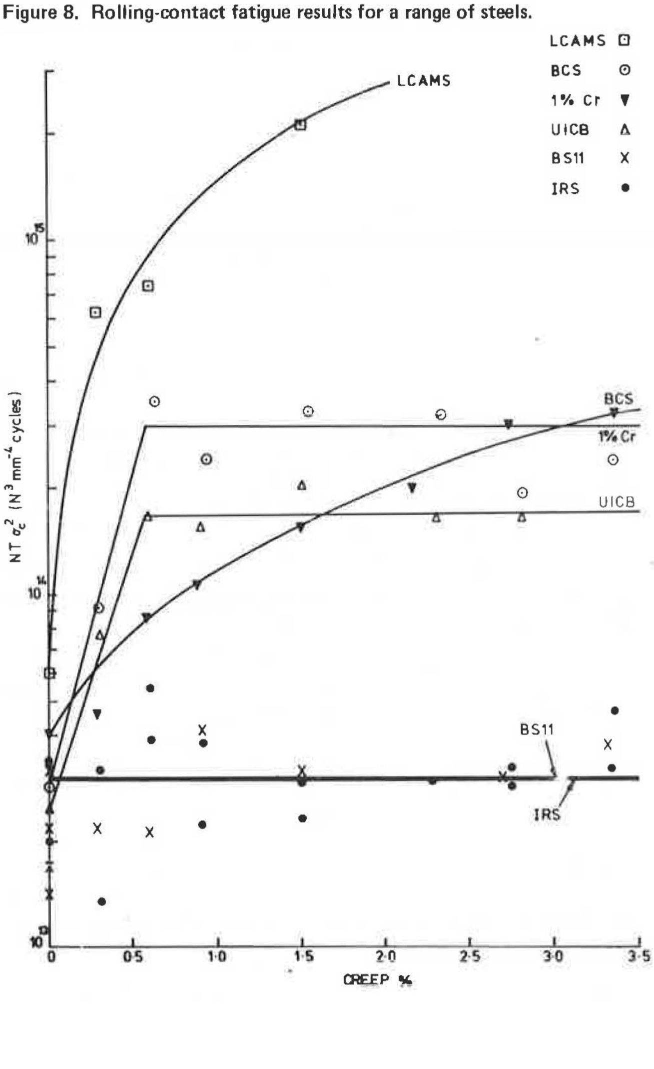


Figure 8. Rolling-contact fatigue results for a range of steels.



Research on rail wear has continued, but a change of direction has taken place. Previous work was aimed at producing semiquantitative relations between laboratory-generated wear data and service experience and attempting to relate wear performance over a wide range of test conditions to a single property parameter. More recent work has been aimed at relating wear to the conditions that exist in the wheel-rail contact zone. A nonlinear curving theory that predicts the forces, creep rates, and contact-zone sizes for a given set of conditions has been developed (13). Current work is aimed at developing an extension to this theory that will lead to wear-rate predictions. Several hypotheses that relate wear rate to traction force T, creep rate γ, and either the contact-zone width 2b or the contact-zone area are under investigation.

A series of laboratory tests performed by using a small-scale Amsler wear-testing machine and a range of creep conditions (1-10 percent) and contact stresses (500-1300 MPa) indicated that two regimes of wear were operating. These regimes were termed mild and severe and were characterized by the debris. Wear rates in the severe regime were found to be dependent on both contact stress and creep rate, whereas in the mild regime wear rate could be independent of creep rate. Correlations were then attempted between wear rates and expressions such as $(T\gamma^n/2b)$. In the mild-wear regime, where there was no interdependence on creep rate, no correlation was obtained. In the severe-wear regime, when combined wear rates (the total wear rate of both rollers) were considered, it was found that neither of the two parameters—$(T\gamma/2b)$ or $(T\gamma^2/2b)$—gave a satisfactory relationship. However, when the value of $n = 1.5$ was substituted, a better relationship was obtained (see Figure 9).

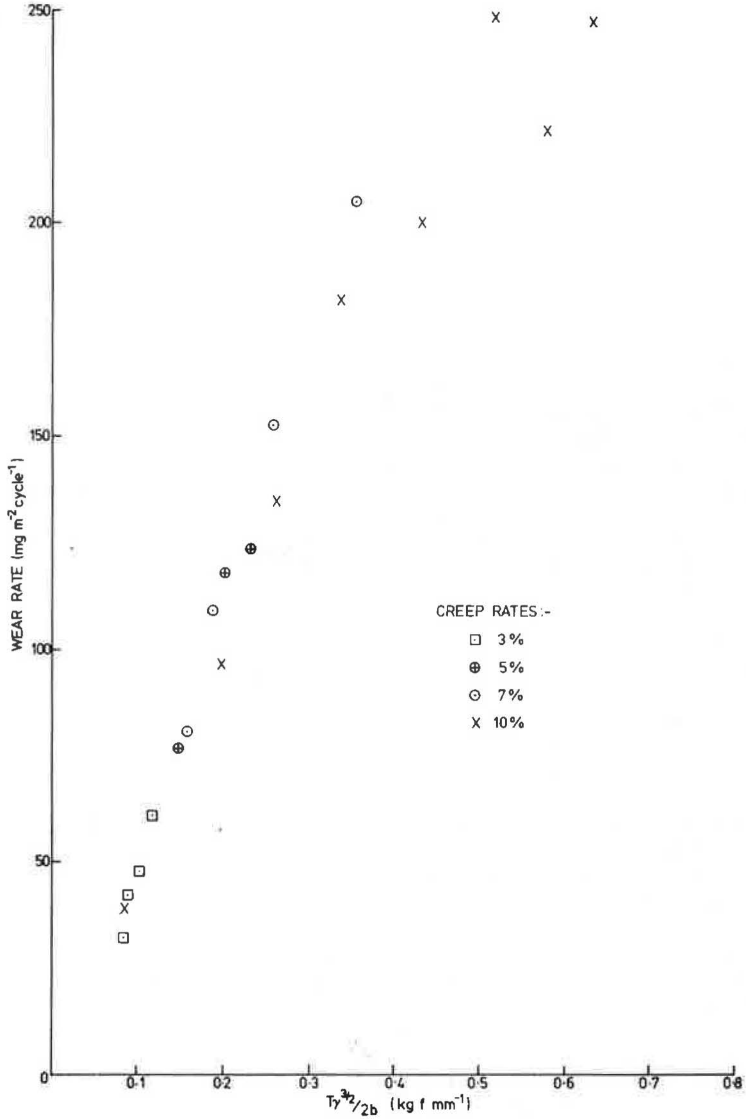
Currently, the effect of contact-zone geometries on wear for given contact stresses is unknown, and therefore the general validity of the wear parameters cannot be established. Further work will investigate this geometry effect and also explore whether the relations apply to other rail steels.

Toughness of Rail Steels

The toughness of steels depends on ambient temperature and rate of load application and, to a large extent, the effects of these two parameters are interchangeable. The lowest toughness can be expected at the highest rate of loading and the lowest ambient temperature. It has been estimated that for BR these are 100 GPa/s and -15°C, although clearly there will be rare occasions when these values will be surpassed. The rate of load application corresponds to the impact of a wheel flat.

Most steels show a transition in toughness from a lower to an upper plateau as the temperature rises.

Figure 9. Wear rate versus ($T\gamma^{3/2}/2b$).



Most pearlitic rail steels, including the UIC grades and 1 percent chromium steel, are on the lower plateau at normal ambient temperatures and slow rates of loading so that the reductions in toughness caused by higher loading rates and lower temperatures are not very large, although they can be significant. When a new, tougher rail steel is produced, however, it is usually found to be in the transition region so that loading rate and temperature are significant. The fracture-tough rail steel developed jointly by BR and the British Steel Corporation is in this category (12).

The effective toughness of rails is affected by residual stresses and also by the size of the specimen. As a result, to measure toughness it is now the practice to test both small specimens and full-section rails. To generate the necessary high loading rates for full-sized specimens, a drop-weight test facility is used. Before testing, a notch and a fatigue crack are formed 6 mm deep across the head of the rail. The rail is tested in bending with the head in tension to simulate wheel-flat loading on a rail with a head defect. Strain gauges are used to identify the load at which the crack begins to run. The drop-weight test facility has also been used in the laboratory to simulate the dynamic loads caused by a wheel flat.

WELDING OF RAILS

Procedures

Considerable effort has been applied to improving the service performance of welds in rails (14). BR uses three basic weld procedures. Flash-butt and Thermit welding are used for producing butt joints, and build-up repair is being increasingly applied because of increases in the number of defects in the rail running surface and the rising cost of replacing rails. In this method, the defects are usually repaired by grinding and building up the surface with metal arc-welding deposit.

Causes of Defects

Until comparatively recent times, the only rail steel used on BR in considerable quantity was the 710-MPa grade. In the two butt-welding processes this material presents few problems; failures are largely confined to Thermit welds. A survey of all Thermit weld failures carried out in 1971 and 1972 indicated that 80 percent of all failures were attributed to operator errors. These failures resulted from (a) lack of fusion, generally in the foot and lower web area (50 percent of all failures); (b) hot tears, usually in the head area (7 percent of all failures); and

Table 2. Composition of commercial rail steels.

Grade	Minimum UTS (MPa)	Composition (percentage by weight)					
		Carbon	Silicon	Manganese	Sulfur	Phosphorus	Chromium
Normal-quality BS 11	710	0.45-0.60	0.05-0.35	0.95-1.25	0.050 max	0.050 max	-
UIC grade A	880	0.60-0.75	0.50 max	0.80-1.30	0.050 max	0.050 max	-
UIC grade B	880	0.50-0.70	0.50 max	1.30-1.70	0.050 max	0.050 max	-
One percent chromium	1080	0.68-0.78	0.35 max	1.10-1.40	0.04 max	0.03 max	1.00-1.30

(c) porosity (6 percent of all failures).

Lack-of-fusion defects are considered to result from poor rail-end preparation, inadequate rail gap, use of oxidizing flame during preheating, and cold additions to the Thermit reaction. Hot tears emanate from premature release of rail tensors, or clamp slip on rail tensors, and porosity results from the use of damp or contaminated molds or luting sand. All of these can be attributed to operator deficiency. As a result of the investigation, a major change in the Thermit process was introduced on BR. From an aluminothermic quick-welding process referred to as the SmW process, which requires 6-7 min of preheating, BR went to a quick-welding process called the SkV process, which requires 1-2 min of preheating, thus reducing the operator dependency involved in preheating [these processes are derived from a German company and marketed by Thermit Welding (GB), Ltd.] The other aspects of operator deficiencies were covered by retraining and adequate supervision of welders in the field. The performance of the SkV weld is currently being evaluated by a further survey of weld failures. Supplies of Thermit consumables are rigidly inspected under a BR specification that ensures a regular supply of consistent consumables.

The failure rate of flash-butt welds is much lower than that of Thermit welds: 0.1/1000 compared to 1.0/1000 for 710-MPa steel. However, investigation of flash-butt-weld failures indicates that 80 percent can be attributed to welding machine and post-weld-treatment deficiencies (40 percent of failures result from incomplete fusion and 40 percent from postweld treatments such as weld trimming and arc spots). Some of the faults can be remedied by good housekeeping; incomplete-fusion defects, however, arise because of machine malfunction or incorrect machine settings that are either electrical or mechanical in action. This type of defect becomes more evident in the welding of higher-strength rail steel.

Higher-Strength Steels

The non-heat-treated, higher-strength rail steels—UIC grades A and B and 1 percent chromium—rely on higher alloy content to develop the higher strength and wear resistance that are their desirable features. However, the higher alloy content also reduces the weldability of these steels, which may require special processes and procedures to produce satisfactory butt and repair welds. Table 2 gives the composition ranges of commercially available rail steels.

UIC Grade A

UIC grade A steel relies on an increase in carbon content to raise its strength level to the 880-MPa minimum value specified, compared with the 710 MPa for normal-grade steel.

Flash welds made with the settings used for normal-grade steel gave good results. Metallographic sections of the heat-affected zones (HAZs) were fully pearlitic in microstructure, and the mechanical properties were satisfactory. Some flat spots—i.e., small oxidized areas that are the remains of arc craters—have been found on the weld line. These can initiate fatigue cracking and subsequent complete transverse fracture of the welds and must therefore be minimized by optimization of the machine controls.

Thermit welds were also found to be satisfactory with fully pearlitic HAZ microstructure and good mechanical properties. Samples put in track showed some "cupping" (local wear) in the weld zone, but this was attributable to the use of a Z80 Thermit portion—i.e., one with 795-MPa tensile strength—whereas a Z90 (880-MPa) strength would now be recommended as matching the rail steel properties.

UIC Grade B

UIC grade B steel derives its extra strength from a 0.1 percent increase in carbon content and an increase of 0.45 percent in manganese content compared with the normal-grade steel. The manganese has a considerable effect on the transformation time, which has to be allowed for during welding. Segregation of manganese aggravates this problem and can lead to the formation of small bands of hard microstructures in HAZs.

In flash welding this steel, difficulties were encountered in optimizing the flash-welder control settings to (a) reduce the flat spots on the weld center line and (b) obtain consistent mechanical properties in commercial production. Flash welding in FB 113A section rail with the optimized machine settings gave cooling rates that were generally slow enough to avoid the formation of hard microstructural phases, although isolated cases of these phases were observed. To minimize the occurrence of flat spots and to maintain good mechanical properties, it was necessary to set machine controls to give (a) an adequate power to flash off the rail, (b) a low enough voltage to avoid deep craters, (c) a high acceleration to avoid oxidation, and (d) a correct flashing distance.

It was also found to be necessary to carry out the investigation on full 18-m-length rails since a considerable variation in results was obtained when welds were made with "shorts", apparently because of power losses through earth leakage. It was concluded that with adequate care and supervision satisfactory welds could be produced in this grade of steel with the current flash-welding machines but that future machines should incorporate feedback control systems.

For the Thermit welding of UIC grade B steel, it was necessary to ensure that a good sound weld with a matched weld metal and nonhardened HAZ was produced. The soundness of welds was attained by use of the SkV short-preheat Thermit process (15), and a Z90 (880-MPa tensile strength) portion was used to give a weld metal that matched the rail in tensile strength and wear resistance. Cooling rates of welds made in FB 113A section rail by using this process were measured at about 20°C/min between 800°C and 400°C. Considered in relation to the continuous cooling transformation diagram for this steel, this would appear to give adequate protection against undesirable hard phases in weld HAZs. However, since these cooling rates were determined under ideal laboratory conditions whereas in-service ambient rail temperatures could be much lower, the effect of muffle cooling was determined. An insulated muffle placed over the welded joint after weld trimming (completed 6 min after weld tapping) reduced the cooling rate from 20°C/min to about 10°C/min in the 800°C-400°C range. Welds produced by using this process and proportion and muffle cooling were free from defects and had good mechanical properties. Further work must be done on the effect of adverse weather conditions and the use of muffle cooling.

One Percent Chromium Steel

The high tensile strength (1080 MPa) of 1 percent chromium steel is generated by the use of as much as 0.78 percent carbon and the addition of as much as 1.4 percent chromium; the alloy additions again had to be allowed for in welding such a steel.

Flash welding was carried out, and machine control settings similar to those used for UIC grade B steel were found to produce welds of good quality. However, the normal cooling rate for flash welds (46°C/min in FB 113A section) produce hardened microstructures in the weld HAZ. Several procedures for retarding the cooling rate were investigated. Direct postheating in the welding machine reduced the cooling rate; about 4 min of postheating was required to produce a satisfactory cooling rate. Muffle cooling was found to be inadequate, but a special flame heating rig in which the joint was heated for 5 min retarded the cooling rate to about 28°C/min. This last treatment gave fully pearlitic microstructures in the HAZs and gave the best and most satisfactory mechanical properties. This procedure reduces delay time in production by not occupying the welder as in direct postheating.

For the Thermit welding of this steel, the SkV short-preheat process is used, and a special high-strength portion has been developed by Thermit Welding (GB), Ltd., to produce matched weld properties. Metallographic examination and mechanical testing indicated that fully pearlitic HAZ microstructures were obtained. However, as with the UIC grade B steel, the recommended practice in service is to use muffle cooling to counter possible effects from adverse weather conditions.

Repair Welding of Rails

Normal-quality rails that have isolated defects in the railhead are often repaired on BR by using the metal arc process, and codes of practice have been issued for the repair of defects such as isolated wheel burns and squats. Work is continuing to extend the procedures to cover the higher-strength rail steels and is aimed at determining preheat levels, the suitability of consumables, preparation geometry, welding techniques, and finishing methods.

Work is also being carried out to explore the use of semiautomatic processes for rail repair, and an evaluation of welding machines and consumables is currently under way. A longer-term objective in this area is the development of fully automatic machines for the repair of railhead defects.

NEW DEVELOPMENTS IN RAIL STEELS

The higher-strength pearlitic steels discussed previously are likely to be sufficiently resistant to wear and fatigue to satisfy BR needs for plain line rails in the foreseeable future. However, the loading environment is more severe in switch and crossing work, and further improvements in rail steels are needed.

BR currently uses a range of crossing types. For the severest locations, cast austenitic manganese steel (AMS) crossings are used. For the less severe locations, the crossing types listed below are used:

1. Bolted AMS crossings (machined from FB 113A section AMS rails),
2. Bolted BS 11 normal-grade crossings (machined from FB 113A section rail), and
3. Semiwelded BS 11 normal-grade crossings (the vees are produced by electroslag welding FB 113A section rail, and wings are attached by bolting).

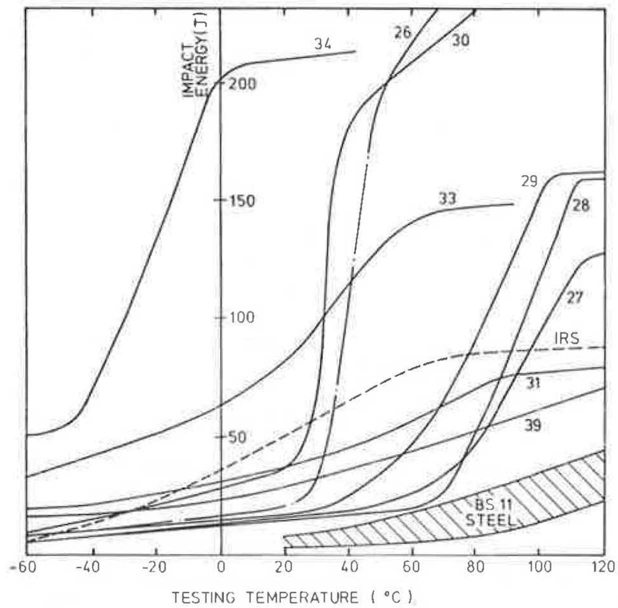
These various types of crossings generally perform adequately in service but have deficiencies of various kinds that could be overcome or minimized by material or fabrication changes. Cast AMS crossings are difficult to produce without casting defects, which lead to structural failure, and the alloy in current use is virtually unweldable. Furthermore, rolled AMS crossings are prone to fatigue cracking that results from the damage produced when the bolt holes are drilled. BS 11 normal-grade crossings deform quickly in heavy-traffic locations because of inadequacies in material strength. There is a need, therefore, for crossing materials that can be produced without defects and are weldable and of adequate strength. Two material developments have led to new fabrication procedures and modifications to existing fabrication procedures.

The problems of casting defects and poor weldability with AMS arise from the high coefficient of expansion and the thermal instability of the conventional Hadfield alloy. The coefficient of expansion of the alloy is much higher than that for the pearlitic rail steels; this, coupled with the narrow range of freezing and the long, narrow castings needed for crossings, results in gross shrinkage cavities and hot tears in the finished product.

The original Hadfield alloy has a carbon range of 1.1-1.4 percent and is used in the water-quenched condition. Subsequent heating above 300°C in welding processes results in carbide precipitation and severe embrittlement of the otherwise high-toughness alloy.

The testing of an experimental series of alloys of varying carbon and manganese levels (16) has shown that they are thermally stable—i.e., there is very little carbide precipitation and resultant embrittlement, providing the carbon level is below 0.9 percent. The decrease in tensile strength properties that results from this reduced carbon content can be offset by increasing the level of manganese. The work has indicated that good mechanical properties and thermal stability can be achieved within the composition ranges 0.7-0.8 percent carbon and 14-17 percent manganese. This composition is now referred to as low-carbon austenitic manganese steel (LCAMS). Commercial quantities of rails have been produced in this alloy, and flash and Thermit welding procedures and consumables have been developed for butt welding of rails in CWR. The high coefficient of expansion of this alloy still limits the use of long, welded rails to locations that have small variations in rail temperature, such as tunnels. However, in the manufacture of crossings, the improved weldability of the alloy lends itself to the production of (a) semiwelded crossing vees and (b) shorter-cast crossing centers with welded-on legs. The development work on semiwelded vees, which is being pursued jointly with Thomas Ward (Railway Engineers), Ltd., is nearing completion, and crossings that incorporate such vees will shortly be installed in track for service evaluation. The production of short-cast crossing centers will relieve the casting defect

Figure 10. Charpy impact curves for bainitic steels.



problem considerably, and welding techniques for the attachment of legs have been developed. Trial track installations will be made for service evaluation.

The remaining problem in the use of LCAMS is the need to weld it to pearlitic rail steels. The alloy contents of the two steels result in the formation of very brittle phases when fusion welding is attempted. Practical solutions are still being sought.

The limitations of LCAMS prompted a search for alternative materials with the desirable properties of LCAMS, such as high strength, impact resistance, and fracture toughness, but without the undesirable properties (the material would have an acceptable coefficient of expansion and could be welded to pearlitic rail steels). The work of Irvine and Pickering (17, p. 292) indicated that such properties could be obtained in the as-rolled condition by an air-cooled, low-carbon bainitic steel. These structures are achieved by the suppression of the ferrite-pearlite transformation, which is best achieved by additions of molybdenum and boron.

Although some data were available on the mechanical properties of some low-carbon molybdenum-boron steels, no systematic study of the effect of carbon, manganese, and chromium on mechanical properties had been carried out. These alloy additions depress the bainitic transformation temperature and thereby improve the tensile properties. An experimental molybdenum-boron alloy series with suitably varying levels of carbon, manganese, and chromium has been produced and tested (18). Tensile strengths as high as 1525 MPa have been achieved, and the fatigue limits were all higher than those of normal-grade steel. The Charpy impact curves were generally much higher than those for normal-grade steels and, in some cases, higher than those previously developed for fracture-tough rail steel (see Figure 10). Work is currently proceeding to evaluate the extent to which these properties can be maintained in commercial production.

CONCLUSIONS

Although the main trend of rail research must be toward improved steels and a better choice of steels and operating practices, it is clear that many problems are still inadequately understood and as such difficult to quantify. It is clear that in the future greater priority must be given to understanding the wear and fatigue mechanisms that act at the surface of rails and how these mechanisms interact. Until this is done, the practicality of defining an optimum steel for a particular set of circumstances must be questionable.

ACKNOWLEDGMENT

We wish to acknowledge the British Railways Board for permission to present and publish this paper and to acknowledge the many contributions made by our colleagues in the Metallurgy, Track, and Structures Section and Strength of Materials Section of the Research and Development Division. We would also like to acknowledge the contribution to the research program that is being made by several companies involved in track component manufacture and maintenance, namely, the British Steel Corporation, Thermit Welding (GB), Ltd., Thomas W. Ward (Railway Engineers), Ltd., Edgar Allen Engineering, Ltd., and Speno International SA.

REFERENCES

1. D. L. Bartlett. Experiments on the Stability of Long Welded Rails. British Transport Commission, London, 1961.
2. C. O. Frederick. The Lateral Strength of CWR. Proc., Symposium on Railroad Track Mechanics and Technology, Princeton Univ., Princeton, NJ, April 1975.
3. G. Samavedam. Buckling and Post-Buckling Analysis of CWR in the Lateral Plane. British Railways Board, Wilmorton, Derby, England, R&D Tech. Note TS 34, Jan. 1979.
4. G. Samavedam and G. Hunt. A Three-Dimensional Analysis of Elastic Stresses in Rails. British Railways Board, Wilmorton, Derby, England, R&D Tech. Note (in preparation).
5. E. I. Danilenko and E. Moras. Measurement of Horizontal Forces on Rails. Die Eisenbahntechnik, Vol. 27, No. 6, 1979, pp. 228-231; British Railways Board, Wilmorton, Derby, England, Translation 79/07/13, 1979.
6. C. O. Frederick. Effect of Wheel and Rail Irregularities on Track. Heavy-Haul Railways Conference, Perth, Australia, Sept. 1978.
7. S. G. Newton and R. A. Clark. An Investigation into the Dynamic Effects on the Track of Wheel Flats on Railway Vehicles. Journal of Mechanical Engineering Science, Vol. 21, No. 4, Aug. 1979, pp. 287-297.
8. Behaviour of the Metal of Rails and Wheels in the Contact Zone: Composition of the Different Stress Conditions in the Wheel/Rail Contact Zone. Office de Recherches et d'Essais, Utrecht, Netherlands, C53, Rept. 7, April 1972.
9. H. O. Asbeck and M. Heyder. Residual and Straightening Stress in New Rolled Rails. Eisenbahntechnische Rundschau, Vol. 25, No. 4, 1977, pp. 217-222; British Railways Board, Wilmorton, Derby, England, Translation D/77/11/15, 1977.
10. P. Clayton. Rail-Grinding Experiments at Rugby. R&D Division, British Railways Board, Wilmorton, Derby, England, Tech. Rept. Met 5, Dec. 1978.
11. P. Clayton. Vertical Wear of Rails. R&D Division, British Railways Board, Wilmorton, Derby, England, Tech. Note FM 55, Jan. 1976.
12. K. Morton and others. Rail Steels. ASTM, Philadelphia, Special Tech. Publ. 644, Nov. 1976.
13. J. A. Elkins and R. J. Gostling. General Quasi-

Static Curving Theory for Railway Vehicles. International Union of Theoretical and Applied Mechanics Symposium, Vienna, Austria, Sept. 1977.
14. M. E. Ashton. Thermit Welding of Rail Steels. Railway Engineer, Vol. 2, No. 3, May-June 1977.
15. J. T. Dyke and M. E. Ashton. Evaluation of the SkV.F Thermit Welding Process for the Welding of U.I.C. Wear-Resistant Rail Grade B. R&D Division, British Railways Board, Wilmorton, Derby, England, Tech. Memorandum Met 31, May 1978.
16. A. Faley and D. S. Hoddinott. The Development of a Weldable High-Manganese Steel. R&D Division, British Railways Board, Wilmorton, Derby, England, Tech. Note FM 67, Aug. 1975.
17. K. J. Irvine and F. B. Pickering. Low-Carbon Bainitic Steels. Journal of Iron and Steel Institute, Vol. 187, 1952.
18. G. M. Pell. The Initial Development of a Bainitic Steel for Crossings. R&D Division, British Railways Board, Wilmorton, Derby, England, Tech. Note Met 13, Dec. 1977.

Publication of this paper sponsored by Committee on Track Structure System Design.

Note: The Transportation Research Board does not endorse products or manufacturers. Trade and manufacturers' names appear in this paper because they are considered essential to its object.

Dilemma of Direct-Fixation Fastening Systems

W. R. Hamilton

A short, nontechnical discussion of the shortcomings of present direct-fixation fastening systems is presented. The state of the art is reviewed, and some suggestions are made on corrective solutions to existing problems.

The phenomenon of the modern railroad track is a mixture of ingenuity, experience, and long trial and error. From the early years of the rail bolted to the flat stone sleepers to the modern integrated structure, there has always been a delicate balance between the mechanical and track design improvements. The problem, however, has always been the same: how to keep the train on the track and provide a low-maintenance running surface. The balance between the track and the train was radically tipped by the dieselization of the railroads and the economics of the maintenance of way so that the track no longer could support the loading forces or provide longevity of running surfaces.

This problem will ultimately be solved as problems in the railroad industry have always been solved. Fortunately, the industry now recognizes the folly of deferred maintenance and acknowledges the track-train relationship as a total system that must be treated as a whole if all of its parts are to operate successfully.

Today, with aid from the federal government, there are literally hundreds of research programs being undertaken in the academic and scientific community. The interface between rail and wheel is being studied; ballast, subgrade, and tie performance is being analyzed; and rail metallurgy is at the moment extremely important. Research in all of these areas can add dramatically to our fund of knowledge and greatly assist in future development of the railroad system.

One area included in these studies, and in my opinion a most important one, is the interface between the tie and the rail, the rail-fastening system. It is not the intent of this paper to analyze mathematically or theoretically the performance of this area of the structure but merely to point out, in general terms, the effect of the fastening system on the performance of the total track structure.

In the broadest sense, the rail-fastening system can be defined as a device that can accurately position two rails with respect to each other and with respect to the earth below them. More specifically, rail-fastening systems are generally considered to concern the method used to affix the rails to the ties. Further, this device should have the capability to selectively absorb or transmit the various loads imposed on the track structure by the rail-wheel contact of passing trains. Without this capability, the remainder of the structure may be subjected to damaging forces that will require additional maintenance and perhaps shorten the life of the structure. However, without the input rail-wheel forces, the structure will also remain relatively inert. We may therefore assume that the fastening device will experience only those forces generated statically by thermal changes in the rail and dynamically by the wheel rolling over the rail.

If one conceptualizes in three dimensions a wheel moving along a rail, it can be seen that many forces are working in all three planes, the longitudinal, the vertical, and the lateral. There are three longitudinal forces. One is the static or thermal force, and the other two are dynamic and are caused by the wave action of the track and the drafting and braking forces of the train. The lateral force created by the wheel has now become a major concern in overturning rail and wheel climbing. The vertical forces are created in a downward direction by the wheel load and converted into an uplift force through the wave action of the track. Imposed on this is the vibration effect caused by the wheel on the rail. This effect has been much ignored because most vibration is short lived and of a greatly varying nature in terms of amplitude and frequency. It does, however, occur in the longitudinal, vertical, and lateral planes; in the lateral plane, it occurs more in curves than in tangent track.

Quantifying the various track loads is beyond the scope of this paper. Available data on the magnitude of longitudinal track loadings, for example, are highly variable. Each researcher has made measurements that he or she believes to be correct but that rarely agree with those made by fellow resarchers. In a personal survey, track engineers were asked what was the maximum static longitudinal pull-apart load in their track. The answers received ranged from 445 to 2670 kN [100 000-600 000 lbf (100-600 kip)], which indicates that we really do not know. On the other hand, it also points out that measurements made in one area are not necessarily always valid in other locations. At the present time, therefore, we use state-of-the-art numbers derived from the conventional tie plate, anchor, track spike concept in designing other fastening systems to suit specific needs. By designing to meet a given longitudinal restraint based on experience, without actually knowing the specific longitudinal load involved, we can assume that the system will work.

In present systems, however, there are two areas that are not only critical but are also being overlooked. These are the uplift of the track and the transmission of the intermittent vibrations through the fastener system to the substructure.

As a basis for discussion, I would go back to work done by Talbot (1) at the University of Illinois under the auspices and with the cooperation of the American Railway Engineering Association (AREA). This work is outstanding, since it did not depend entirely on mathematical hypothesis but rather included actual track measurements. By using existing theory, Talbot determined that the rail performed as a continuous beam on elastic supports and that the vertical movement of the crosstie varied with the elasticity of ties, ballast, and subgrade. This he defined as the track modulus. In addition, Talbot's investigators measured rail stresses and found that the passage of a wheel caused compressive stresses in the upper fibers of the rail but also that tensile stresses equal to approximately half of the compressive stresses occured in the same location before and after the passage of the wheel. This stress reversal indicates that the rail is rising from its neutral position in front and in back of the wheel.

To join these two dissimilar facts, it was further shown that the higher the track modulus is, the greater is the stress reversal or uplift. As the track becomes softer or less rigid and the load is distributed further, the total downward deflection becomes greater and at some point the stress reversal ceases, only to become a variation in the compressive load. Zero, however, is never reached. From a practical standpoint, the worst condition would occur on a continuous surface, such as a rigid slab with a very rigid rail fastener. The initial failure of the fasteners at the Kansas slab test track and in earlier tests at the Pere Marquette track are two good examples of uplift.

How does the conventional railroad track handle this particular problem? The line spike on the tie, even though driven home, in most cases will shortly back off sufficiently to allow the rail to breathe on the tie or essentially to "float". In the conventional structure, if this does not occur, the tie lifts in the ballast, creating the "pumping" phenomenon, which is most undesirable.

Generally speaking, accelerations greater than 100 Hz have been discounted, since most vibration is quickly damped and of short duration. Recently, however, renewed interest has been shown in the frequency bands and their harmonics and repeating patterns. The highest activity is found where the continuity of the rail surface is broken or where flat spots on wheels impact the railhead. These impacts have been measured at greater than 80 *g* and 800 Hz.

Probably one of the reasons vibrations have been disregarded is the lack of transmission through a wood tie into the substructure. When the rail is "floating" on the conventional structure, vibration is only transmitted periodically when the rail base is in contact with the tie plate, and what is transmitted is damped by the wood. Changes in load, speed, and location change the magnitude of the vibratory effects. In some initial results in field studies being run by Portec, Inc., it appears that vibration effects on various types of structures are the same in the rail—through the fastening and into the tie regardless of track stiffness. The structure of the tie, however, has a great effect on the amount of the forces transmitted to the ballast and subgrade.

Having isolated three of the acting forces in the rail-tie interface—i.e., longitudinal loads, uplift, and vibration—it can now be shown what effect these three forces have on the fastening problem and why what is being done today may be more detrimental than beneficial to the track system.

Until the 1950s, asking a track designer to fasten a rail and tie together was unthinkable, but that is precisely what is being advocated today through the introduction of the direct-fixation systems. The concept comes from Europe and was introduced originally in the 1930s in the form of the "GEO fastener". This was abandoned after some test installations because of cost, complexity, and poor performance in track (pumping ties). In the 1950s, the introduction of concrete ties revived this concept. Since a spike cannot be driven in concrete, it was necessary to provide some other method. These fasteners have now begun to appear on wood ties as well.

In the conventional track construction of tie plates, spikes, anchors, and wood ties, the loading factors are absorbed from rail to tie through a number of separate pieces. The uplift is generally ignored, since the rail is not constrained vertically. The plate is constrained to the tie so that other loads are transferred through the plate and spikes to the tie and the anchors transfer the longitudinal loads from the rail to the tie. Vibration is somewhat damped in the wood.

In the fastening system, a single device reacting against the rail base produces a "toe load" that is suppose to accomplish restraint longitudinally and vertically. This is done through the use of various types of springs that are supposed to make the fastener flexible. They are instead semirigid. This concept is in itself a design enigma. On the one hand, this spring device must have the capability to restrain longitudinal rail running, and therefore this toe load must be high. Conversely, the spring must be flexible enough to allow the rail to move upward without affecting the tie and ballast. This, then, requires either a lower toe load and/or a spring with a very low spring rate. Unfortunately, it is next to impossible to achieve both with a single device, and generally the compromise is toward higher toe loads because the designer feels that longitudinal restraint is more important than the uplift effect.

In defense of these systems, and remembering the European origin, testing has shown that the original systems lack longitudinal restraint but that they do allow the fastener to breathe with the rail. With lighter wheel loads, the uplift is minimized and the accelerations applied to the system are reduced. This, coupled with the lighter spring system, allows the rail to move upward so that the uplift is absorbed into the fastener and the tie or ballast is not disturbed. Unfortunately, track parameters in North America are quite different from those in Europe, and most fasteners fell far short of the necessary loads.

In order to meet the longitudinal requirements, most manufacturers modified their systems to provide a stronger spring but also a more rigid fastener. Most marginally meet the longitudinal parameters but have now increased the spring rate, which, coupled with the larger steel section, has changed the characteristics of the system. In addition, the higher wheel loads increase the uplift forces.

Now, with the higher spring constant, greater uplift loads, and the same rate of loading, the stiffer spring cannot absorb the force rapidly enough so that the load is transferred to the tie. The uplift force being larger than the weight of the tie, it tends to move the tie vertically. Even though these excursions are relatively small, they are the beginning of ballast migration, center binding, pumping, and fines rising to the surface.

Under real-world operating conditions, one wheel with a single wave does not occur. There are many wheels, complex wave motions, and a myriad of bands and intensities of vibration traveling in some six directions at any given point under the train. As a train approaches a given point, the rail preceding the lead wheel begins to vibrate. Anyone who has walked along continuously welded rail can attest to the audible "singing" of the rail sometimes long before a train appears. It must also be remembered that this vibration is traveling in the opposite direction back of the wheel and that this pattern is repeated for the second, the third, and all wheels. The same is true of loads on the rail. Over a given fastener, the loadings and the frequency spectrum of the vibration are constantly changing, and this causes sharp changes in the motion of the spring. This confusion may cause the spring to stagnate so that the loads are transferred directly to the tie; it may cause the spring to resonate, which may account for the massive movements seen in track ties; or, because of differences in mass, it may cause the tie and the rail to move in opposing directions vertically.

Let us look for a moment at the effect of fasteners on the design of concrete ties. Since its introduction, this concept has changed considerably until the tie we know today, which meets AREA requirements, is a 340-kg (750-lb) behemoth designed according to criteria far in excess of loads measurable in track performance. In addition, when the present structure is used with concrete, it must be ballasted with coarse, hard ballast since the concrete tie abrades ballast. Initially, it was thought that concrete ties could be put on centers as high as 77 cm (30 in), but today, although the ties are designed to the 77-cm parameters, they are rarely spaced beyond 61.5 cm (24 in).

Conversely, there is an installation in which concrete ties are installed on 77-cm centers in volcanic ash ballast. The tie is designed to much lower standards and weighs only 255 kg (560 lb), considerably less than the AREA tie. This installation has been in track since 1970 on a class 6, 113-km/h (70-mile/h) track and has seen more than 181 million gross Mg (200 million gross tons) of traffic. During this period, the track has been surfaced once and gravel ballast has been added. The track has performed satisfactorily. By comparison, adjacent tie installations that have the semirigid type of fasteners have not done as well with many problems of tie skewing and pumping.

The common denominator of these installations on concrete is the fastener. There is undoubtedly general agreement that today's concrete tie designs are successful and that they have performed admirably at the Facility for Accelerated Service Testing. Successful or not, the fact remains that they do move under dynamic loading. Isn't the reason for the hard-ballast requirement perhaps the fact that the movement of the tie grinds or crushes the softer ballast, creating fines that ultimately foul the ballast or migrate to the surface? If there were no tie movement, would this ballast requirement be the same? In addition, one might ponder the fastener effect on design and spacing. Obviously, in view of the tie movement on the semirigid fasteners, the dynamic requirements of the concrete will be much higher than in the free-floating concept, where the tie is more or less inert in the ballast. From such evidence, one might be tempted to arrive at the wrong hypothesis. It must be agreed that there are many unknowns at this point and much to learn.

The direct-fixation system has one big advantage, and that is the simplicity of its parts. This, of course, is reserved for the drive-on type of system. Usually, the removal of one part on each side of the rail frees the rail from the tie. This, compared with removing anchors and pulling spikes, makes the new system most attractive. This would be particularly true in curved sections, where heavy traffic loads require frequent transposition or relay of the rail. For this reason, direct fixation has found its way into wood-tie application. Use of these ties mounted on tie plates allows the rail to be removed without pulling spikes or removing anchors. The only questions I would ask are the following: Since the uplift reaction has now been transferred to the tie surface, how will the interface between tie plate and tie react? If a 340-kg (750-lb) concrete tie lifts in the ballast, what will happen to the 68-kg (150-lb) wood tie?

At this point, one might think the direct-fixation fastener has very little merit in the present-day track system. Quite the contrary, this system has much to offer in stabilizing the track and decreasing maintenance. The fact that what we know today presents problems and the technology is incomplete should only add spice to the challenge of resolving the problem.

This paper only serves to point out what we consider to be shortcomings in current devices in the hope that users will recognize these areas and either be prepared to live with them or demand better fastening systems. The fastener designer should recognize the effect the designs have on the overall performance of the track and at the very least be prepared to provide corrective solutions and ultimately to design to meet the requirements.

There is one bright spot in this situation that will alleviate, if not correct, the present problem. Remember that most of the present systems are adaptations of lighter systems that lose much of the original performance when stiffened. North American suppliers are now beginning to recognize the merits of direct fixation, and new devices are entering the marketplace that are designed to meet the heavier requirements unhindered by past designs of lighter systems. These systems may make the best compromise for all the known loadings and, although they are not the ultimate solution to the total problem, they are a great improvement to track performance.

At this point in time, there is a great deal about fastener performance that is unknown. To unlock these secrets from the track will require a return to basics by the researcher. This means the accumulation of data from the structure itself. In areas where questions are unanswered, modeling will not suffice. This information gathering is not a short-term proposition but will require several years. My own organization has been doing this for three years, and we have barely scratched the surface. We find that much must be done in instrumentation, for without good means of measurement the data are either incomplete or useless. Data should be taken under every possible condition, and analysis done with extreme caution.

The answer is there and will ultimately become obvious. In achieving this goal, each step must be taken carefully and fully documented. Through this method, we stand to improve design and performance rather than hinder it.

REFERENCE

1. A. N. Talbot. Stress in Railroad Track. Proc., AREA, Vol. 19, 1918, pp. 873-1062; Vol. 21, 1920, pp. 645-814; Vol. 24, 1923, pp. 297-453; Vol. 26, 1925, pp. 1081-1245; Vol. 31, 1930, pp. 69-336; Vol. 35, 1934, pp. 66-308.

Publication of this paper sponsored by Committee on Track Structure System Design.

Comparison of Performance of Wood-Tie Fasteners at FAST

Howard G. Moody

The results of experiments with wood-tie fasteners at the Facility for Accelerated Service Testing are reported. Since the beginning of the use of 91-Mg (100-ton) freight cars, there has been an increasing problem with wood-tie fasteners. The resulting high axle loads have caused an increase in the deterioration of wood ties from spike killing and tie-plate cutting and have taxed the cut-spike fastener to the limit in preventing rail rollover and wide gage. The objective of the wood-tie fastener tests at the Facility for Accelerated Service Testing is to find an alternative to the cut spike that would alleviate some of the problems that have occurred in revenue service. Two test cycles have been completed, and a third is currently being run. In the first test, an excessive amount of rail wear, attributed to high flanging forces and a lack of effective lubrication, resulted in two rail transpositions. The rail was regaged each time, which eventually "spike killed" the ties. In the second test, in the elastic-clip segments, a large number of the hold-down fasteners failed, which resulted in wide gage. This led to a redesign for the current test to incorporate four hold-down fasteners, twice as many as in the second test. The results have not yet demonstrated that there is a wood-tie fastening system that will perform better than the cut spike. The results of the third test, however, may change this conclusion.

For the past 10-15 years, the increasing dynamic forces imposed on track by the 91-Mg (100-ton) cars on North American railroads have, in many instances, created conditions in which the traditional cut-spike fastener has not performed well. In particular, the heavy wheel loads in curved track have resulted in rapid rail wear, which has led to frequent transposition, relay, and replacement of rail. These maintenance activities and heavy axle loads have resulted in wide gage from spike-killed ties. In addition, it is apparent that on sharp curves the cut-spike fastener is less than optimal in preventing rail rollover and the looseness of the fastening system that can lead to tie-plate cutting.

Many railroads, including the Atchison, Topeka, and Santa Fe Railway Company, the Bessemer and Lake Erie Railroad Company, and the Canadian National Railway, have recently been testing alternative fastening systems in main-line track in curves that would resist gage widening and rail rollover, eliminate respiking, and restrain the rail longitudinally without the use of rail anchors. The systems that are being tested include a variety of elastic fasteners and compression clips that are designed to alleviate the problems associated with the cut spike.

At the Facility for Accelerated Service Testing (FAST), similar fastening systems and others that are not currently being used in revenue service are being tested to find the elusive alternative to the cut spike as a wood-tie fastener.

BACKGROUND ON FAST

A general description of the FAST track, the consist used, and the test conditions is given in the paper by Steele and others elsewhere in this Record. The diagram of the FAST track shown in Figure 1 pinpoints section 7, where wood-tie-fastener testing is being conducted.

A record of the traffic load accumulated to July 1979, before a major summer rebuilding, is shown in Figure 2. The traffic load to that date was 381 million gross Mg (419 million gross tons) (traffic loadings throughout this paper are in gross megagrams and gross tonnage). This rapid accumulation of load greatly accelerates the wear and fatigue life of the track components. Track components that might normally experience 27 million Mg (30 million tons) in revenue service in one year will accumulate 136 million-182 million Mg (150 million-200 million tons) at FAST.

In addition to the heavy traffic loads at FAST, other factors should be taken into consideration in evaluating FAST results in relation to revenue service. These factors are

1. The lack of flat wheels in the consist;
2. The speed range of 64-72 km/h (40-45 miles/h);
3. The 5- to 7.6-cm (2- to 3-in) underbalanced conditions in the curves;
4. The short test duration and relatively mild, dry climate, which tend to reduce environmental effects; and
5. The grade and track configuration of each test section with which comparisons are being made.

TEST DESCRIPTION

The wood-tie-fastener tests are conducted in section 7 of the FAST track, which consists of a 303-m (1000-ft) long, 5° reverse curve with a 0.07 percent grade that has been divided into test segments. Since the test section is a reverse curve and, as shown in Figure 3, the train speed through the curve is generally between 68 and 72 km/h (42 and 45 miles/h), the curve is one of the most severe environments at FAST. Histograms of the vertical and lateral forces produced by the consist operation are shown in Figures 4 and 5. These data were taken on the high rail. In this curve, with a 5- to 7.6-cm (2- to 3-in) underbalanced condition, the high rail has higher dynamic loads than the low rail. The mean lateral forces are greater for the lead axles, which is almost always the case in curved track.

The means and standard deviations for vertical wheel load (Figure 4) are as follows (1 kN = 224.8 lbf):

Rail	Mean (kN)	Standard Deviation (kN)
High	145.19	33.89
Low	122.73	27.09

The means and standard deviations for lateral wheel load (Figure 5) are as follows:

Axle	Mean (kN)	Standard Deviation (kN)
Lead axle, lead truck	40.57	14.72
Trail axle, lead truck	23.26	6.58
Lead axle, trail truck	46.44	14.01
Trail axle, trail truck	21.13	7.03

The primary intent of the wood-tie-fastener tests is to determine qualitatively and quantitatively the ability of different fastening systems to restrain the rail laterally and to prevent rail rollover. A secondary objective is to determine what types of fastener systems minimize tie-plate cutting and "spike killing" of ties in the test section. Each test sequence—and there have been three to date—is expected to last until there are component failures that cause wide gage throughout the test section or until there is a deterioration of the ties that results in a substantial number of ties being replaced.

One of the overall objectives of the FAST experiments is to permit comparison and evaluation of as many different components as possible while adhering to and maintaining statistically sound practices. In the wood-tie-fastener experiment in section 7, everything except the fasteners has remained the same from one test to another, with few exceptions. The ballast, curvature, grade, and continuously welded rail (CWR) have remained constant. Except for one case in the third test, the ties have also stayed the same.

Two sequences of tests have already been performed, and a third sequence is in progress. The past tests were used partly to establish which fastener components warranted further investigation and under what conditions further testing was required. The first test ran from September 1976 to September 1977 and accumulated a traffic load of 0-122.347 million Mg (0-134.582 million tons). The second test ran from November 1977 to November 1978, accumulating 122.434-326.007 million Mg (134.678-358.608 million tons). [In reality, the second test was effectively terminated in July 1978 at 249 million Mg (274 million tons).] The third test began in January 1979 at 326.55 million Mg (359.21 million tons).

All fastener types have been and will be compared with the performance of 35.5x20-cm (14x7.75-in) 1:40 American Railway Engineering Association (AREA) tie plates with 15x1.6-cm (6x0.625-in) cut spikes and conventional rail anchors.

In the first test, there were five 61-m (200-ft) segments. The second and third tests had (have) ten 30.5-m (100-ft) segments. The location of a particular test component within the curve may change, but there will always be at least a single test control segment that has the standard AREA plates and cut spikes. In general,

Figure 1. FAST track.

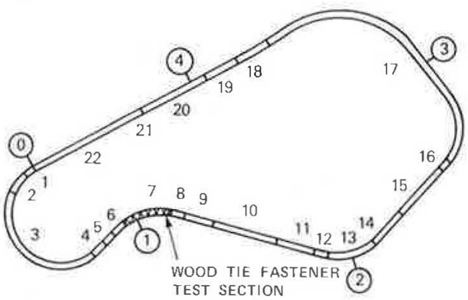
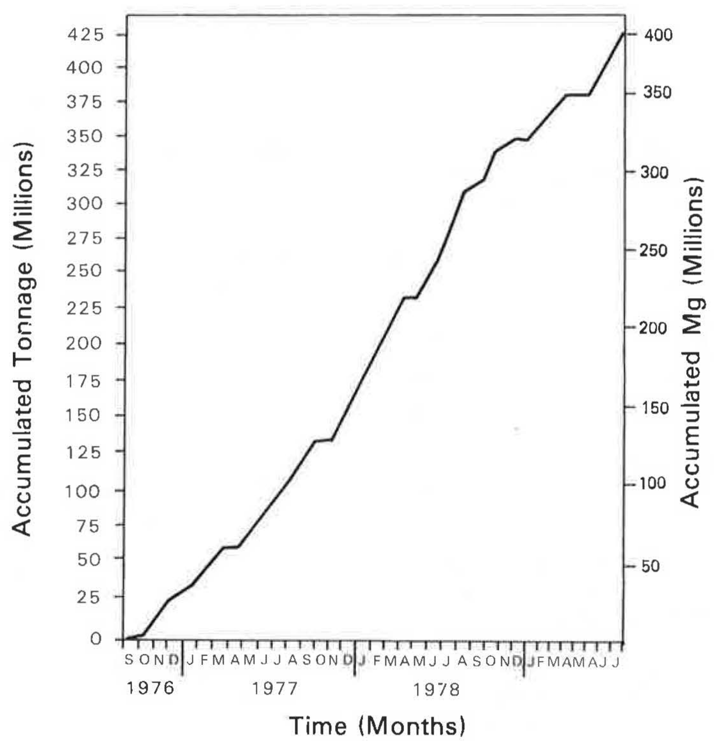


Figure 2. Accumulated traffic load.



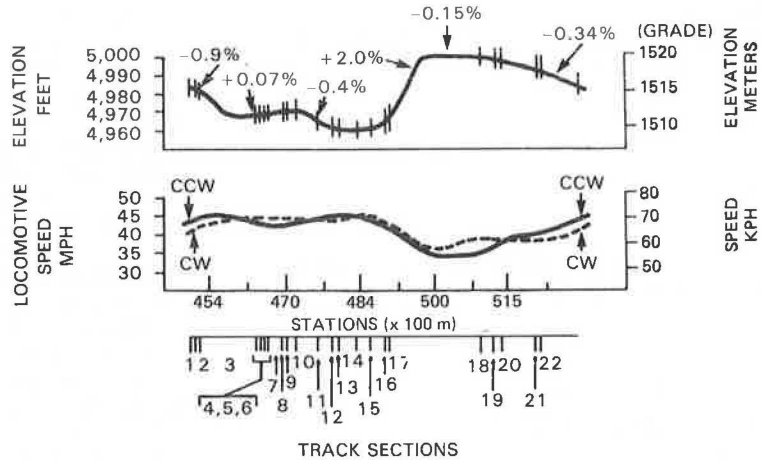


Figure 3. Track and locomotive-speed profiles.

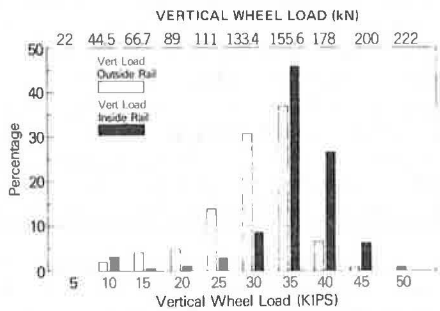


Figure 4. Vertical wheel loads.

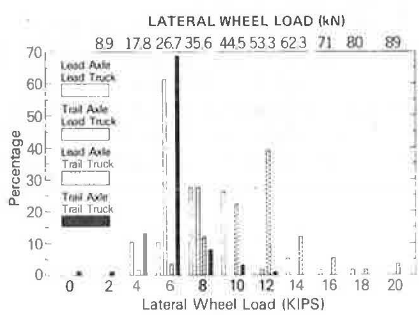


Figure 5. Peak value for lateral wheel loads.

all new test components are installed at one time to prevent differentiating effects caused by changes in operating variables. Failed components are replaced in kind from the same lot of components.

The experiment design is intended to limit the number of controlled variables that affect fastener performance. Ties are selected for the test from the same species and if possible from the same treatment lot. All of these ties are new. The ballast cross section is constant throughout the test section. As a result, there should be uniform track support conditions throughout the test section. To achieve this, a high degree of track maintenance is performed in section 7 so that anomalies of track geometry that could affect fastener performance will not occur.

A 30.5-m (100-ft) test segment contains 61 or 62 ties on 49.5-cm (19.5-in) centers. Each segment can be considered to consist of a central test zone isolated from the adjacent segments by transition zones. If a sample of 30 ties in the central zone is accepted as statistically big enough to permit the drawing of valid inferences about a large population, and if each transition zone contains 6 ties, then the 30.5-m test-segment length is more than adequate. No measurements are taken in the transition zones.

Table 1 gives the layout for each of the three test sequences. The ballast is slag, and in the past two tests the rail has been 69.5-kg/m (140-lb/yd) RE CWR.

The properties of the test components that help to determine fastener performance are spring rate or resiliency, toe load, and fatigue strength. Because of the short test segments, toe load or fastener longitudinal restraint was not measured.

Several static and dynamic measurements have been taken in section 7 to meet the stated objective. These measurements are (a) static gage and gage point wear, (b) track geometry (gage only), (c) tie-plate cutting, and (d) vertical track modulus.

The static gage and track-geometry-gage measurements are similar in that they both measure the net gage widening under little or no load. The measurement of static gage and gage point wear is taken by using a standard gage bar to measure the gage and a snap gage device to measure the wear on the gage corner of the high rail. By subtracting the wear on the rail from the standard gage, a true value of track gage is obtained. These measurements are made at specified intervals in five locations in each segment. The measurement of track geometry gage is taken from the track geometry data as measured daily by the EM-80 track geometry car. It is a continuous measurement and serves as a check on the static gage measurement.

The measurement of vertical track modulus is used to define track support conditions by using the beam-on-elastic-foundation theory (because the equation is formulated in U.S. customary units, no SI equivalents are given), in which

$$k = \sqrt[3]{P^4/64EIY^4} \tag{1}$$

where

k = track modulus (lbf/in^2),
P = applied load (lbf),
EI = stiffness of the rail (lbf/in^2), and
Y = deflection of the rail under load (in).

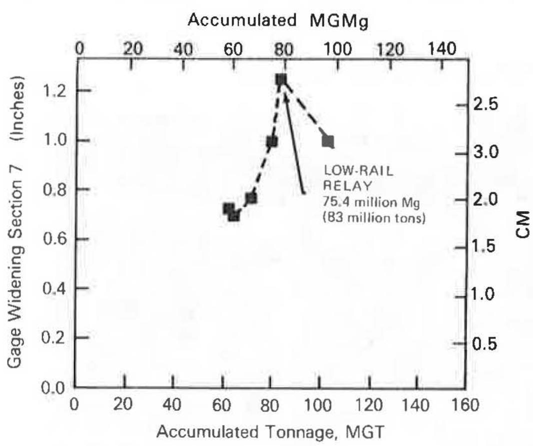
The dynamic measurements are (a) wheel-rail loads, including rail vertical load V and rail lateral load L; (b) dynamic gage widening; (c) lateral railhead deflection H; and (d) lateral rail-base deflection B. The dynamic measurements are used to characterize the dynamic load environment in each track segment. At one location in each segment, wheel-rail loads and lateral

Table 1. Layout of FAST section 7 for three wood-tie-fastener tests.

Segment	Tie No.	Type of Fastener
Test 1[a]		
1	001-123	35.5x20-cm 1:40 AREA A plate, four 15x1.6-cm cut spikes, box anchored every other tie
2	124-246	35.5x20-cm 1:40 AREA A plate, 15x1.6-cm cut spikes (two line) and lock spikes (two hold-down), box anchored every other tie
3	247-370	35.5x20-cm 1:40 AREA B plate, two compression clips, two 15x1.6-cm cut spikes
4	371-492	Plate and elastic clip, two lock spikes
5	493-616	35.5x20-cm 1:40 AREA A punch plate, 15x1.6-cm cut spikes (two line), screw spikes (two hold-down), box anchored every other tie
Test 2[b]		
1, 10	001-061, 553-616	35.5x20-cm 1:40 AREA B plate, three 15x1.6-cm cut spikes, box anchored every other tie
2, 6	062-122, 308-368	35.5x20-cm 1:40 AREA A plate, two compression clips[b], two 15x1.6-cm cut spikes
3, 7	123-184, 396-428	1:40 elastic-clip plate, two 2.7-cm screw spikes
4, 8	185-245, 429-490	1:40 elastic-clip plate, two 15x1.6-cm cut spikes
5, 9	246-307, 491-552	1:40 elastic-clip plate, two 1.9-cm cone-necked drive spikes
Test 3[c]		
1	001-060	Standard 35.5x20-cm 1:40 A punch plates; five 15x1.6-cm cut spikes: on gage side, one plate holding and two line; on field side, one plate holding and one line; every other tie box anchored
2	061-121	1:40 elastic-clip plates; four 0.8-cm screw spikes, one each corner of plate; one elastic clip each side of rail, diagonally opposite
3	122-182	1:40 elastic-clip plates; four 0.8-cm screw spikes, one each corner of plate; one elastic clip each side of rail, diagonally opposite
4	183-243	1:40 elastic-clip plates; four 1.9-cm cone-necked drive spikes, one each corner of plate; one elastic clip each side of rail, diagonally opposite
5	244-304	1:40 elastic-clip plates; four 1.6-cm lock spikes, one each corner of plate; one elastic clip each side of rail, diagonally opposite

Note: 1 cm = 0.39 in; 1 m = 3.3 ft; 1 Mg = 1.1 tons.
[a]All ties were 18-cm x 23-cm x 2.7-m southern yellow pine.
[b]Terminated May 8, 1978, at 198 million Mg.
[c]Ties for segments 1, 3, 4, and 5 are 18-cm x 23-cm x 2.6-m southern yellow pine; segment 2 ties are 18-cm x 23-cm x 2.6-m mixed hardwoods.

Figure 6. Static gage widening.



railhead and rail-base deflections are measured simultaneously at frequent intervals of approximately 23 million Mg (25 million tons). By using the lateral load and the deflection measurements, which are referenced to the tie, the rail and fastener spring rate or stiffness and the fastener's ability to resist rail rollover and gage widening are defined. The dynamic gage-widening measurement determines the widest gage that occurs during one passage of the consist at both the railhead and the rail base. Since it only measures the greatest excursion or spread of the two rails, it reflects a "worst-case" condition. These measurements are taken at five locations in each segment.

With the exception of the dynamic gage-widening measurement, none of the dynamic measurements were taken in the first two tests in section 7.

RESULTS

The first test was terminated in September 1977 after 122.347 million Mg (134.582 million tons) because of wide gage from spike-killed ties. This condition resulted from constant regaging, between 19 million and 29 million Mg (21 million and 32 million tons), during a period when the rail wear was very rapid, and again at 75 million Mg (83 million tons), when the low rail was relayed.

The high rate of rail wear in the first 29 million Mg was a result of a lack of lubrication. In section 7, almost 1.9 cm (0.75 in) of wear in the high-rail gage corner occurred in this early period. At 29 million Mg, the rail was transposed. After another 45 million Mg (50 million tons), the plastic flow on the railhead in the low rail became excessive and rail from the bypass track was relayed into section 7. In addition, as Figure 6 shows, the track gage at this time was rapidly deteriorating because of the regaging done earlier. Consequently, when the rail was relayed, the track was regaged. After another 36 million Mg (40 million tons), the gage became increasingly difficult to maintain and gage bars were applied, which effectively terminated the test.

There were few component failures during the first test, and there was no tie-plate cutting in any of the test segments. Any comparison of the fastener groups in their ability to hold gage was confounded by the constant regaging.

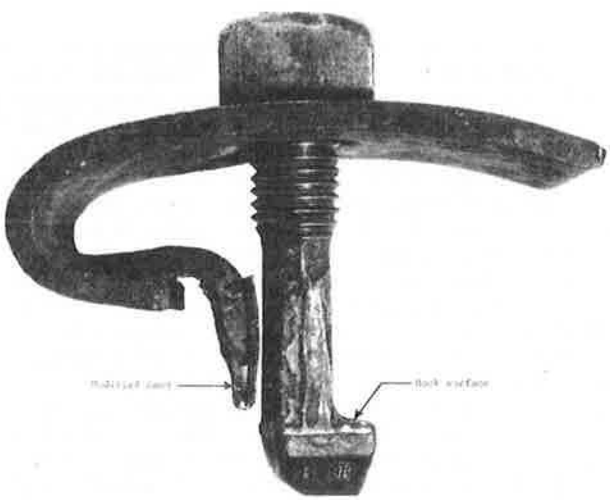
Section 7 was rebuilt during October 1977, and the second test was begun in November 1977 at 122.434 million Mg (134.678 million tons). In this test, the component segment length was reduced to 30.5 m (100 ft), and replications of each segment were included to determine any position-in-curve effects.

The first component test that had to be terminated was associated with segments 2 and 6. The compression-clip bolt (see Figure 7 for installation) loosened, and the clip backed out of the tie plate and came off the rail. Sixteen clips out of 488 were removed by hand on March 9, 1979. In the next two months, several other clips backed out. Since it was difficult to reapply them, it was decided to terminate the test in segments 2 and 6.

A combination of events led to the termination of the test. Because the ties were not prebored, the surface

Figure 7. Compression-clip assembly.

Figure 8. Modified compression clip and tie-plate bolt.



of the clip bolt hook (see Figure 8) did not always fit properly under the tie plate. The 1.9-cm (0.75-in) punched hole in the tie plates used in this test usually had a small radius on the edge of the hole that prevented the clip from fully engaging the bottom of the tie plate. This radius is a normal result of the hole-punching operation when the plates are manufactured. In addition, there was enough dimensional tolerance in the bolt and tang of the clip to allow the bolt to back out of the punched hole in the plate when the punched hole was slightly worn.

Figure 9. Failure of elastic-clip plate.

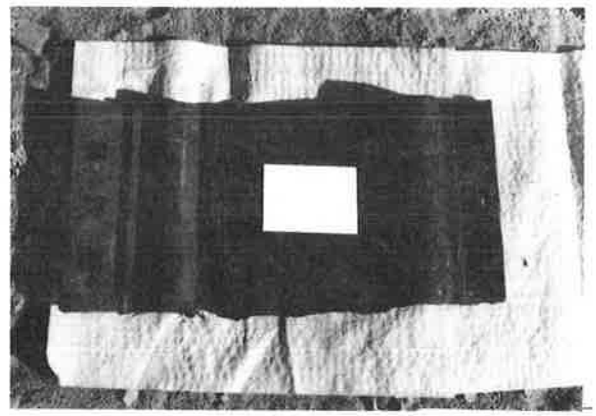


Figure 10. Failure of elastic-clip plate in track.

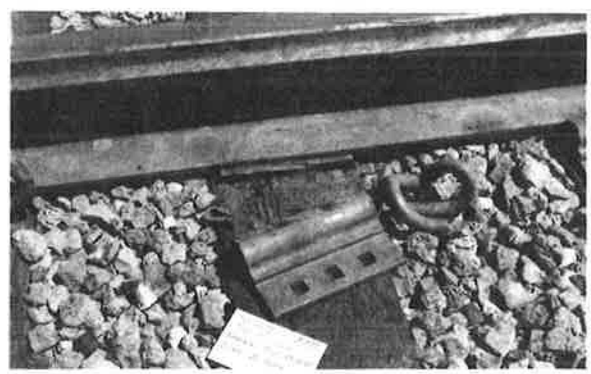


Table 2. Failures of wood-tie-fastener components in second test.

	Elastic Clip Out or Off				Broken Tie Plate		Spike Out 2.5 cm or More or Failed			
	Gage Side		Field Side				Gage Side		Field Side	
Component	Rail	No.	Rail	No.	Rail	No.	Rail	No.	Rail	No.
35.5x20-cm 1:40 AREA plate, three 15x1.6-cm cut spikes, box anchored every other tie	NA		NA		Inside Outside	0 0	Inside Outside	0 1	Inside Outside	0 0
38x20-cm 1:40 elastic-clip tie plates, two 2.7-cm screw spikes, two elastic clips	Inside Outside	2 1	Inside Outside	5 11	Inside Outside	10 2	Inside Outside	151 8	Inside Outside	10 4
38x20-cm 1:40 elastic-clip tie plates, two 15x1.6-cm cut spikes, two elastic clips	Inside Outside	2 5	Inside Outside	6 2	Inside Outside	22 19	Inside Outside	10 2	Inside Outside	4 0
38x20-cm 1:40 elastic-clip tie plates, two 1.9-cm cone-necked drive spikes, two elastic clips	Inside Outside	4 2	Inside Outside	2 1	Inside Outside	6 6	Inside Outside	9 25	Inside Outside	5 18

Note: 1 cm = 0.39 in.

Compression clips were used in the first test in section 7 without the occurrence of any of these problems. However, the clips used in the first test, as shown in Figure 8, had a 0.152- to 0.228-cm (0.060- to 0.090-in) shim tack welded to the tang on the compression clip. The clips used in the second test did not have the extra shim. It is felt that this modification effectively prevented the clips from backing out unless the nut on the bolt was loose enough for the tang to slip out from behind the bolt.

Early in the second test [the first failure occurred

Figure 11. Failed screw spike.

Figure 12. Screw spike out more than 2.5 cm (1 in).

Figure 13. Cumulative rate of screw-spike failures versus traffic load.

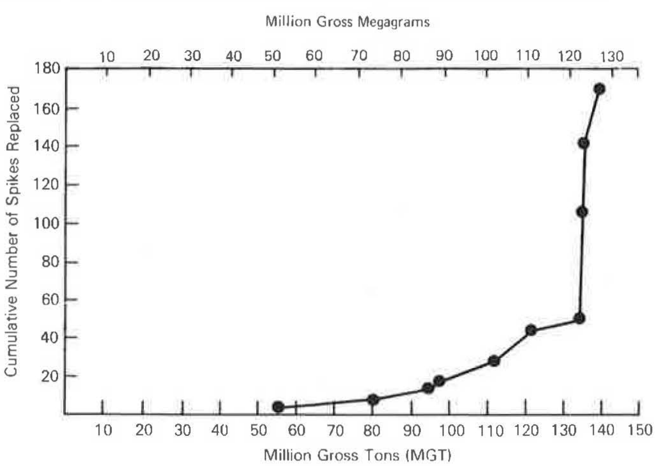


Figure 14. Static gage variation.

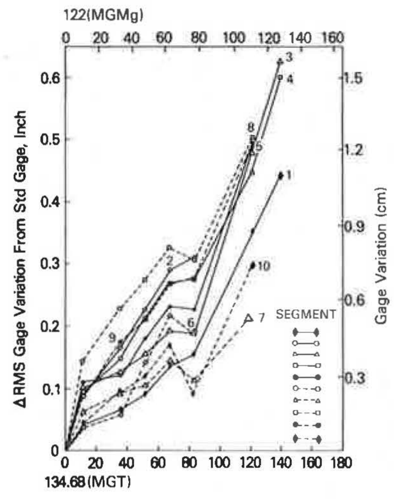


Figure 15. Mean dynamic gage widening.

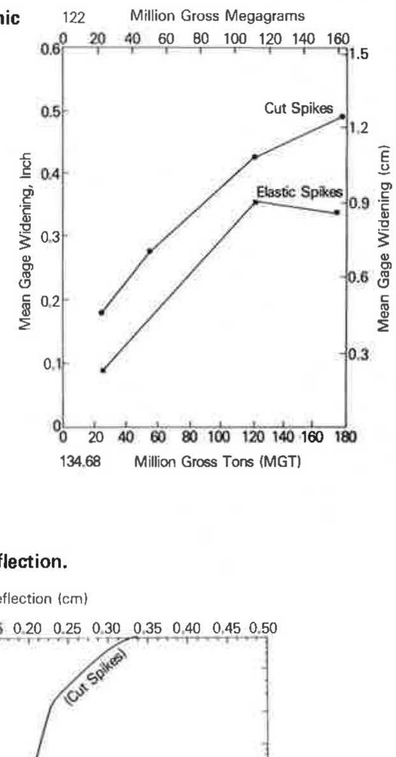


Figure 16. Railhead deflection.

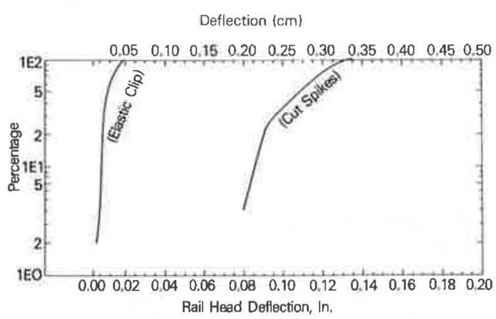
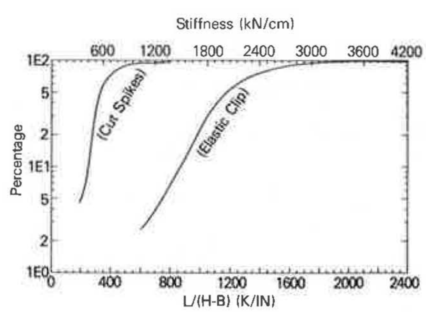


Figure 17. Fastener stiffness.



at 141 million Mg (155 million tons)], a few elastic-clip plates began to fail. These bending failures occurred at the lead edge of the plates where the outside of the rail edge contacted the plate (see Figures 9 and 10). As data given in Table 2 show, 65 of these failures occurred out of a sample population of 732 plates. Forty-one of the failures occurred in cut-spike segments 4 and 8. These were the original plates from the first test. By the time the second test was terminated, they had almost 12 million load cycles on them. There were not enough failures to determine a characteristic curve on the plate but, in contrast, over an accumulation of 213 million Mg (235 million tons), 38 AREA tie plates out of a lot of 22 120 failed.

It is not known whether or not these failures have occurred at all in revenue service and, since the choice of the hold-down fastener may have contributed to the failures, it is difficult to say whether or not this would occur outside of FAST. However, the manufacturer has since redesigned the plate to considerably increase the cross-sectional area in which the fractures occurred. These new plates should perform substantially better.

Beginning at 36 million Mg (40 million tons), the hold-down fasteners in elastic-clip segments 3, 4, 5, 7, 8, and 9 began either to work out of the tie or to fail. A complete tabulation of these problems is given in Table 2. These problems were most pronounced in screw-spike segments 3 and 7. In a period of 127 million Mg (140 million tons), 173 of 484 spikes in these segments either failed (see Figure 11) or came out of the tie more than 2.5 cm (1 in) (see Figure 12). A plot of the failure rate is shown in Figure 13. At about 118 million Mg (130 million tons), the failure rate began to escalate, eventually causing the test to be terminated at 128 million Mg (141.1 million tons).

The majority of the fractured screw spikes, drive spikes, and cut spikes failed in bending about 5-7.6 cm (2-3 in) below the head of the spike. When these failures occurred, the stub end of the spike was driven through the tie and a resin tie filler was used to fill the hole as a new spike.

The result was a deterioration in track gage. Figure 14 clearly shows that, except for segment 7, there was a rapid deterioration in gage after about 73 million Mg (80 million tons) of traffic [200 million Mg (220 million tons) overall]. The dynamic gage widening was also becoming severe, as shown in Figure 15.

The other problem of note in section 7 is elastic-clip fallouts, which are listed in Table 2. These are addressed in another FAST report and will not be discussed here.

It was evident from the results of the second test that the number and type of hold-down fasteners needed to be changed. Consequently, the number of screw and drive spikes was increased to four, and the cut spikes were replaced by lock spikes. In addition, a segment of hardwood ties with screw-spike fasteners was added to duplicate a similar test in revenue service.

Early data taken in section 7 on railhead deflections (see Figure 16) and comparisons of rail fastener stiffness [L/(H-B)] (see Figure 17) indicate a substantial difference in the responses of cut spikes and one kind of elastic clip. Relative railhead movement is much greater with the cut spike, which results in more gage widening, as Figure 15 shows. However, the rigidity of the elastic clip puts a premium on providing adequate hold-down capacity between the tie plate and the tie. In the second test, it was obvious that there were not enough fasteners of this type.

SUMMARY

The results of wood-tie-fastener testing at FAST to date have shown that the alternative elastic clip or compression clip has not performed better than a conventional cut-spike fastener. The dynamic effects of traffic on each system are much different, which indicates that different design considerations must be taken into account. The elastic clip puts a premium on the type and number of hold-down fasteners used to hold the tie plate to the tie, whereas the cut spike allows so much rail movement that there is a greater probability of gage widening and, as has been demonstrated in revenue service, a likelihood of spike-killed ties. Given these basic conditions and the results of the second test, the current test is much more likely than the second test to result in an adequate design alternative to the cut spike.

Publication of this paper sponsored by Committee on Track Structure System Design.

Development of an Analytical Approach to Track Maintenance Planning

A. E. Fazio and Robert Prybella

Current research being conducted in a joint effort by the Consolidated Rail Corporation and the Federal Railroad Administration to develop an integrated maintenance-of-way planning model is reported. To develop a rational plan for maintenance-of-way expenditures, it is necessary to predict the effect of increased "basic" track maintenance on the requirement for "discretionary" maintenance. Basic (routine) track maintenance is performed by small, labor-intensive section and subdivision gangs. Discretionary track rehabilitation is performed by large, mechanized track maintenance gangs that move about the track system. Basic-maintenance gangs generally complete a particular task at a higher unit cost than discretionary-maintenance gangs. The frequency of the discretionary-maintenance cycle varies as some function of the level of basic maintenance—i.e., as the level of basic maintenance is reduced, the interval between discretionary-maintenance cycles is shortened. The limiting case, in which basic track maintenance is restricted to complying with safety requirements, requires the most frequent performance of discretionary maintenance. For various reasons, it is generally desirable to fund basic maintenance at a level greater than this minimum.

One of the most serious problems currently facing America's railroads is the deteriorated track structure. For many years, as profit margins decreased, railroads reduced their operating costs by a somewhat subjective program of deferred maintenance. The absence of an analytically based planning methodology made it difficult to identify much of this deferred maintenance at the time at which it occurred, and this made the overall financial situation of the railroads appear to be better than it actually was. In the recent rail renaissance, there has been an intensive effort by railroads to rehabilitate their physical plant, particularly the track structure.

Total maintenance-of-way expenditures for class 1 railroads in the United States in 1978 exceeded $3.4 billion (1). The apportionment of these funds between "discretionary" and "basic" maintenance was generally not accomplished by using a standard planning model; instead, money was budgeted and spent on the basis of subjective techniques that varied from railroad to railroad. Once this upgrading is completed, it will be necessary to apply a maintenance-of-way planning methodology that will normalize track maintenance cycles and clearly and objectively identify incidences of deferred maintenance.

An analytical planning technique that is capable of fulfilling these needs is currently being developed in a joint effort by the Federal Railroad Administration (FRA) and the Consolidated Rail Corporation (Conrail). Because the model is currently under development, its theoretical basis and the specific tasks prerequisite to the formulation of a working model are presented here by using hypothetical data, where required, for illustrative purposes.

CURRENT PRACTICE IN TRACK MAINTENANCE

Track maintenance is generally grouped into two broad categories: (a) rehabilitation, or discretionary maintenance, and (b) routine, or basic, maintenance.

The basic-maintenance track gang evolved from the "section gang", which, in its original form, is nearly extinct in the United States. The section gang predated the development of sophisticated machinery for track renewal and repair and consisted of a dozen, or fewer, men who were assigned to perform all required maintenance on a specific, limited [approximately 50-km (30-mile)] section of track. Contemporary basic-maintenance forces were created by placing section gangs in trucks and enlarging and combining their territory of responsibility. The development of mechanized rehabilitation gangs changed the role of these basic-maintenance forces.

Consider, for example, tie renewal. Before the advent of the mechanized tie gang, subdivision gangs were required to replace all deteriorated ties in their territory. They would therefore replace all ties in their territory over the life cycle of the ties. Now, however, with a periodic visitation by a mechanized tie gang, the section gang need only replace a portion of the ties that deteriorate and could even totally abstain from spot replacement of ties. Since mechanized gangs replace ties at a lower unit cost than do subdivision gangs, it might appear to be preferable not to use subdivision forces at all to replace ties. But other factors, including (a) the fact that the rate of deterioration of a good tie depends on the percentage of bad ties around it and (b) the limited availability of mechanized tie-renewal machinery, dictate that some tie renewal be performed on a routine basis by subdivision (basic-maintenance) forces.

There currently exists no analytical method of apportioning maintenance-of-way activities, such as tie renewal, between discretionary- and basic-maintenance operations. To develop an integrated approach to track maintenance planning, the model must identify the impact that a given level of basic-maintenance funding has on the required frequency of the discretionary-maintenance cycle.

CHARACTERISTICS OF MAINTENANCE GANGS

The most common discretionary-maintenance gangs are generally classified as rail, tie-renewal, or surfacing gangs and are characterized as follows:

1. They use track maintenance machinery—e.g., tie inserters and tie-spiking machines—extensively.
2. They are a large force of relatively inexperienced and unskilled labor, generally 40-80 men, performing repetitive tasks in an assembly-line type of operation.
3. The work must be planned well in advance. Because of the size of the gang and the fact that most of the track machinery is rail-bound, the gang cannot readily relocate. A summer's rehabilitation is generally planned the preceding winter.
4. They are designed to replace or repair a specific track component, such as ties, and modify or replace other components only to the extent necessary.
5. The gang has the capability to replace or repair a particular track component at the minimum unit cost.

Basic maintenance is performed by small gangs that may operate anywhere in a particular jurisdiction. These gangs have the following characteristics:

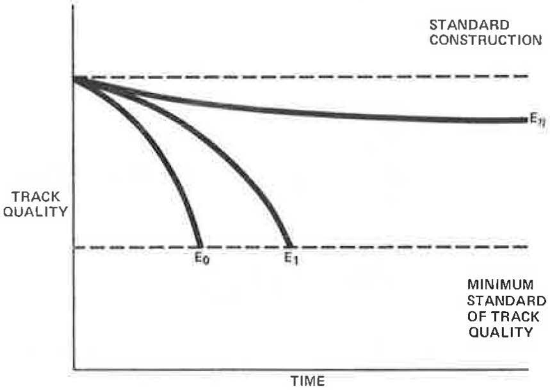
1. They make very limited use of track machinery and instead rely heavily on hand tools.
2. They are a small force, generally 3-10 men. Each individual, however, generally possesses a higher degree of skill and has had more varied track-work experience than personnel on rehabilitation gangs.
3. They have relatively high mobility. They may, in fact, perform maintenance in locations up to 80 km (50 miles) apart during the course of the day. Their mobility is a result of their limited use of track-bound machinery. Because of this, their work need not (although it may) be programmed in advance.
4. They are equipped to replace or repair a wide variety of track components.
5. The replacement or repair of a track component by basic-maintenance gangs is generally done at a higher unit cost than when it is done by discretionary-maintenance gangs.

Basic-maintenance gangs complement discretionary-maintenance gangs in that they (a) protect against catastrophic occurrences, such as a derailment, by giving prompt attention to discrete track failures, such as a broken rail; and (b) control the rate of deterioration of the track during the interval between service by the discretionary-maintenance gang.

QUANTIFICATION OF TRACK CONDITION

Prerequisite to the formulation of an analytical model for track maintenance planning is the development of a mathematical measure (or measures) of track quality. In general terms, track quality can be defined as the ability of the track structure to meet its functional requirements (2). According to this definition, the type of operations supported by the track will influence the selection of the parameters used to measure quality. Although many railroads have "in-house" estimators of track quality—such as Conrail's "condition index" (3)—existing estimators either lack universality or re-

Figure 1. Hypothetical track deterioration curves.



quire the collection of excessive amounts of field data to calculate the parameter for a given track segment.

A current FRA-sponsored research project is attempting to formulate measures of track quality, called "track quality indices" (TQIs), which can be derived either totally or primarily from data collected by automatic track geometry cars. Parameters that can be measured by these cars include gauge, cross level, warp (rate of change of cross level), and alignment (4). The FRA-owned cars are capable of recording measurements every 0.8 m (2 ft) at speeds up to 242 km/h (150 miles/h) and carry on-board computers so that a computer tape of the collected data, in addition to the strip chart supplied to maintenance-of-way personnel, is available at the conclusion of the survey. Each of these parameters of track geometry has a unique distribution over a section of track. The current FRA-sponsored project is investigating the suitability of various statistics of these distributions as TQIs.

Although it would be desirable to develop the TQIs solely from data that can be collected automatically, it will probably be necessary to supplement these parameters with other track data, such as bad-tie counts. This might be avoided by developing the capability to estimate track modulus—the ability of the track to resist deflection under load—by using automatic track geometry cars.

Operating conditions will affect the selection of TQIs. One should expect different TQIs, for instance, to be applicable to tracks that support 80-km/h (50-mile/h) freight traffic and tracks that support 160-km/h (100-mile/h) Metroliner service.

The current FRA-Conrail project is focusing on operating conditions that are common to many North American freight railroads: mixed freight trains of 50-100 cars operating on conventional track (wood tie and stone ballast) at speeds of 64-80 km/h (40-50 miles/h). In Europe, the Office of Research and Experiments (ORE) is also attempting to develop an easily measurable quantifier of track condition based on automatic track geometry cars (5).

DEVELOPMENT OF TRACK DETERIORATION CURVES

The rate of deterioration of track structure is a function of (a) structural parameters, such as ballast type and depth, tie size and spacing, rail weight and cross section, track gradient, and alignment; (b) usage parameters, such as annual traffic load and its wheel-load distribution, and train speed; (c) environmental factors, such as annual rainfall and freeze-thaw cycles; and (d) the amount and type of basic maintenance performed.

Currently, there is no proven method of predicting the deterioration of track as a function of these variables (6, p. 2). The development of a workable set of TQIs will permit the experimental development of track deterioration curves. These curves would show the change in the quality of the track, as measured by the TQIs, as a function of time.

Figure 1 shows hypothetical track deterioration curves; each curve shows the degradation of track quality with time at constant levels of expenditure for basic maintenance E, where $E_i > E_{i-1}$. Various levels of expenditure (E_i) for basic maintenance provide for different rates of track deterioration. For all $E_i < E_\eta$, deterioration to the minimum standard eventually occurs. The minimum would represent, at worst, the FRA standards for each class of track. Since FRA standards are minimum standards, as dictated by safety considerations, many railroads prefer to define their minimum standard at a slightly higher track quality.

In Figure 1, E_0 represents the limiting case of zero expenditure for basic maintenance—i.e., once the track is rehabilitated, almost no maintenance is performed. Even in this case, there will be expenditure for certain required basic-maintenance tasks, such as safety inspections. Curve E_1 represents a slightly higher level of expenditure for basic maintenance. Thus, if E_0 = \$50/year/track-km (\$80/year/track mile), and this was expended for inspections and emergency repairs, E_1 might be \$150/year/track-km (\$242/year/track mile), the incremental expenditure being used for spot tie renewal, tightening joint bars, and other miscellaneous basic-maintenance tasks that serve to retard track deterioration.

At the level of expenditure E_0, the track is usable for some time but deteriorates rapidly. The other extreme, shown in Figure 1 as E_η, represents an intensive basic-maintenance effort. At this level of expenditure, no significant deterioration of the track occurs; this would be representative of track maintenance before the advent of mechanized discretionary-maintenance gangs. Conrail's current unit costs for renewal of one main-line tie by discretionary-maintenance and basic-maintenance gangs, respectively, are approximately \$30 and \$50. Obviously, the level of basic maintenance E_η does not capitalize on the inherent lower unit costs associated with mechanized gangs and is thus not desirable. This leads to the conclusion that the role of basic-maintenance forces should be to regulate the rate at which track deteriorates and not to maintain track at a steady-state condition. Thus, the level of expenditure for basic maintenance should be less than E_η.

Certain parameters that measure a particular aspect of track quality may actually show improvement of the track with use. Track modulus increases as trains are operated over newly rehabilitated track (7). This is attributable primarily to the compaction of the ballast as trains operate over track whose surface has recently been reworked. Even as this compaction is occurring and track modulus is increasing, individual components of the track structure (ties and rails) are continually deteriorating. Thus, although overall track quality increases for a short period of time immediately after the reworking of the track surface, the increase in track modulus caused by ballast compaction eventually comes to an end and deterioration begins.

It is generally agreed that the rate of deterioration of track is a function of its condition and, as the condition worsens, the rate of deterioration increases (8,

Figure 2. Suspended rail joint in good condition.

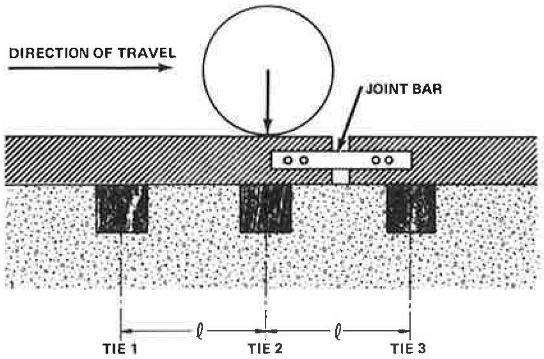
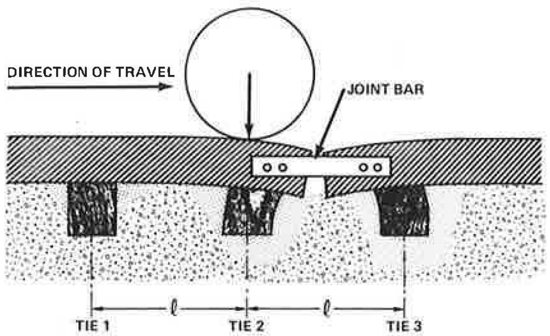


Figure 3. Suspended rail joint in poor condition.



p. IV-40). A qualitative illustration of why this occurs can be obtained by considering the suspended rail joint shown in Figure 2, which supports a point load, the wheel, where shown. A currently used design practice calls for the assignment of 50 percent of the dynamic axle loading to tie 2 and 25 percent of this loading to each of ties 1 and 3 (9). This assumes that the joint bar is stiff, the wheel rolls over a smooth joint, and the rail acts as a continuously supported beam in supporting the wheel load (10).

Although this may be a good assumption for track in perfect condition, use of the track causes the joint bar to loosen. As the joint bar loosens, the rail end begins to act more like a cantilever than a continously supported beam. This causes a greater loading on tie 2 and, thus, accelerated deterioration of this tie and of the ballast beneath it. As the tie and ballast degrade, the structure loses its ability to support the load without significant movement. A differential movement between rails of 0.39 cm (0.125 in) or more is not uncommon and causes rapid deterioration of the rail because of the deformation that results from cold working of the steel. This is manifested in rail-end batter and surface bending of the rail, as shown in Figure 3. Ties 1 and 3 also deteriorate at an accelerated rate because of the impact loading imposed on them when the wheel crosses the joint.

FACTORS THAT AFFECT DETERIORATION

For a given level of expenditure E_i, different deterioration curves will evolve for each variation of the significant structural and usage parameters. For example, Figure 4 qualitatively demonstrates the effect of doubling the yearly gross traffic load while all other parameters remain unchanged. Figure 5 shows the

Figure 4. Track deterioration as a function of traffic load.

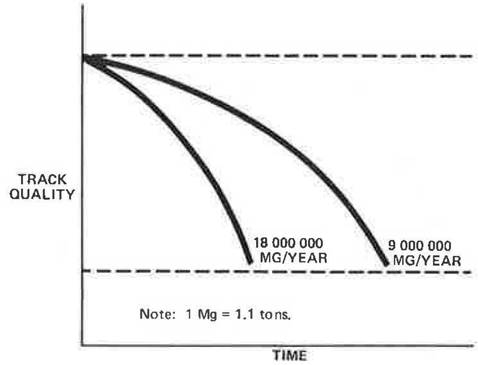


Figure 5. Track deterioration as a function of curvature.

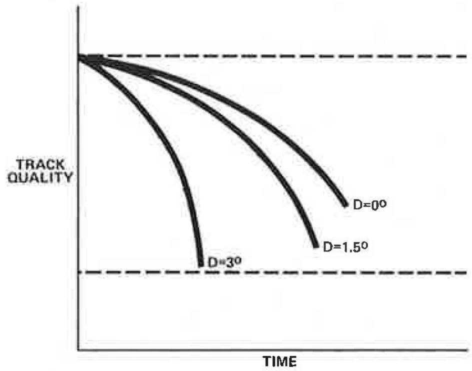
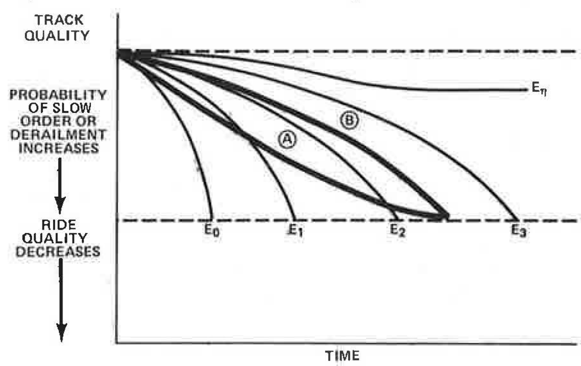


Figure 6. Alternate maintenance strategies.



effect of track curvature on rate of deterioration.

In general, the change in track quality can be expressed as

$$\Delta TQI = f(e, S, T, P, V, M) \quad (1)$$

where

- e = level of basic maintenance;
- S = structural characteristics of the track, such as continuously welded or jointed rail, ballast depth, and subgrade type;
- T = annual traffic load;
- P = wheel-load distribution of the annual traffic load;
- V = train speed; and

M = miscellaneous parameters, such as weather and train consist.

Current research in the FRA-Conrail program is attempting to determine the sensitivity of ΔTQI to each of these factors and to determine which set of factors will account for the bulk (80-90 percent) of track degradation. The functional dependence of the change in track quality on each of these significant factors must then be determined for inclusion in the deterioration model. Graphically, this means that a unique family of deterioration curves must be developed for each variation of the factors that are found to significantly affect the change in TQI.

DEVELOPMENT OF BASIC-MAINTENANCE STRATEGY

Once the track deterioration function is formulated, the TQI can be graphed as a function of time for any given set of structural and usage parameters. Curves that indicate the desired condition of the track, as measured by TQI, at any given time can then be superimposed on this coordinate system. Such curves describe a "basic-maintenance strategy".

Two typical strategies are shown in Figure 6. The selection of a particular strategy may be based on a variety of criteria. A curve such as A in Figure 6 can be selected for certain freight-only lines when it is desirable to keep basic maintenance to a minimum. The presence of passenger service or hazardous materials on a certain route may indicate the selection of curve B, for, as Figure 6 indicates, the probability of "slow orders" and track-caused derailments increases, and ride quality decreases, as track quality decreases. Thus, strategy B allows the track to spend more time in a condition in which ride quality is good and probability of derailment is low.

Note that both strategies must prevent the track from deteriorating beyond the minimum standard and, for purposes of comparison, the minimum standards for both cases are assumed to be identical. In practice, however, the minimum standard for a line with passenger service or hazardous materials might well be fixed at a better quality of track. When the track reaches its minimum, discretionary maintenance should be performed to return the track to the "new" condition. Note that a rehabilitation gang—a tie-and-surfacing operation, for example—does not quite return the track to the as-constructed standard because conventional rehabilitation gangs are not designed to effect a complete renewal of track structure.

The total cost of a given maintenance strategy over a complete cycle of deterioration and renewal consists of the cost of the discretionary-maintenance operation plus the cost of the basic-maintenance strategy. The latter is given by the expression

$$C_A = \int_A e(dt) \tag{2}$$

where C_A is the total cost of basic maintenance for strategy A and e is the expenditure for basic maintenance for an increment of time dt. The integral is computed along curve A.

The total cost of maintenance if strategy B is selected is the cost of rehabilitation plus the cost of basic maintenance for strategy B. The latter is

$$C_B = \int_B e(dt) \tag{3}$$

where the integral is computed along curve B. In the case shown in Figure 6, since both rehabilitation operations are identical and occur at the same time, the total cost of maintenance strategy A will be greater than that for strategy B by the expression

$$\Delta C = \left[\int_B e(dt)\right] - \left[\int_A e(dt)\right] \tag{4}$$

Because an analysis of this type relates the condition of track at a given time, and its rate of change, to the funding level for basic maintenance, it can be used in conjunction with the unit costs for discretionary maintenance to determine the total cost of a desired maintenance cycle. It can therefore be used to evaluate alternate maintenance strategies—for example, the effects of varying the interval on which discretionary maintenance is performed.

This model also identifies the avoidable costs related to maintenance of way, such as the cost of providing commuter service. In addition, it clearly identifies occurrences of deferred maintenance as occasions in which the maintenance strategy allows the track quality to fall below the minimum standard.

EXAMPLE OF USE OF THE MODEL

The following example of the use of this model has been developed by using purely hypothetical data and is included for illustrative purposes only. Some of the terms used in the equations are given in SI. For the most part, however, the equations have been formulated in U.S. customary units, without SI equivalents.

Assume a track quality index Q whose value immediately subsequent to the passage of tie-and-surfacing gangs to tangent, jointed rail track is 100 and whose minimum accepted value for this type of track is 76. Also assume that, for the fixed structural parameters of this type of track, the following three usage parameters are found to account for 90 percent of the rate of track degradation: (a) annual tonnage (T), (b) percentage of tonnage that moves in cars larger than 100 gross tons (P), and (c) average speed (V).

Assume that track quality Q as a function of time t is given by

$$Q = Q_0 - A(t^2) \tag{5}$$

where

$$A = f(T, P, V, e) \tag{6}$$

and where

Q_0 = 100,
t = time (years),
T = annual tonnage (million gross tons),
P = percentage of tonnage in cars larger than 100 gross tons,
V = average speed of trains, and
e = level of expenditure for basic maintenance.

Further assume that it has been shown experimentally that A is determined by the functional relation

$$A = (T/10)(P/50)(V^2/900)(E_0/e) \tag{7}$$

where E_0 is the minimum level of basic-maintenance funding necessary to meet requirements for inspection and emergency repairs.

Now the basic-maintenance budget for a selected unit

Figure 7. Hypothetical track deterioration curves at three funding levels.

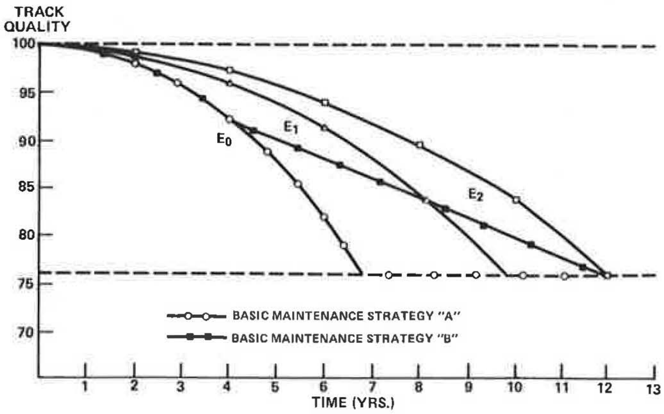
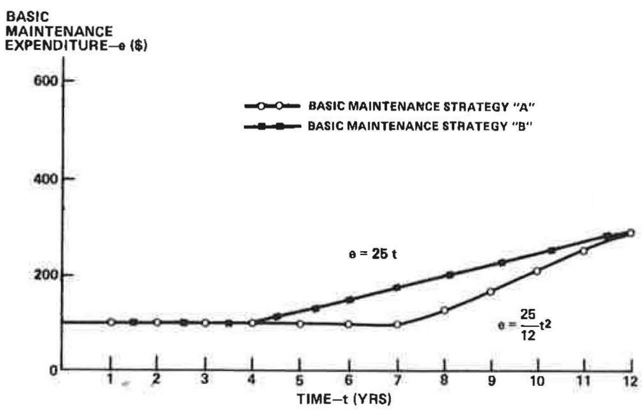


Figure 8. Expenditure for alternative basic-maintenance strategies as a function of time.



of track can be developed. Assume that the track to be budgeted for has the following usage parameters: T = 100 million gross tons/year, P = 25 percent, V = 30 miles/h, and E_0 = \$100/track-km/year. Substitution of these values into Equations 4 and 6 yields

$$Q = 100 - (50/e)\,t^2 \qquad (8)$$

Figure 7 shows deterioration curves for this track at three levels of basic-maintenance funding: E_0 = \$100/track-km/year, E_1 = \$200/track-km/year, and E_2 = \$300/track-km/year. Suppose that the discretionary maintenance required to return the track quality to Q = 100 is to be performed on a 12-year cycle. The expenditures for any basic-maintenance strategy can now be easily calculated.

Consider strategies A and B in Figure 7. Strategy A provides the minimum total basic maintenance for the 12 years. The cost per kilometer of this strategy is as follows:

$$\int_{t=0}^{12} e(dt) = (7)\,(100) + \int_{7}^{12} e(dt) = 700 + \int_{7}^{12} (50/24)t^2\,(dt)$$
$$= 700 + 962 = \$1662/\text{track-km} \qquad (9)$$

Further assume that this route carries commuter trains that must be "slow ordered" when Q is less than 80. Since commuter trains would be slow ordered for six years under basic-maintenance strategy A, another strategy might be preferred.

Strategy B, which is defined mathematically as

$$Q = 100 - (2t) \quad \text{for } t > 4 \qquad (10)$$

and

$$e = E_0 \quad \text{for } t < 4 \qquad (11)$$

allows trains to operate at normal speed for a greater period and would be more acceptable. Note that, each year after the fourth, strategy B requires an increment in the level of basic-maintenance funding. Its total cost is

$$\int_{t=0}^{12} e(dt) = 4(100) + \int_{4}^{12} e(dt)$$
$$= 400 + \int_{4}^{12} (25t)dt$$
$$= 400 + 1600 = \$2000/\text{track-km} \qquad (12)$$

Figure 8 shows basic-maintenance expenditure for strategies A and B as a function of time. The total expenditure for each alternative is the area under its curve in this figure.

The avoidable costs of track maintenance associated with the commuter service (strategy B) can be graphically identified on an annual basis. If this service had been provided at the bequest of a government agency, e.g., an operating authority, these costs could be billed to that agency.

This model could also be used for the costing of freight service. If, for example, a particular shipper's traffic were to double the percentage of annual tonnage moved in cars that weigh more than 91 gross Mg (100 gross tons), the additional costs incurred could be readily identified by adjusting the value of the coefficient A in Equation 6.

SUMMARY

An analytical, integrated methodology for railroad track maintenance planning would be of use to the railroad industry. The following tasks are currently being researched jointly by Conrail and FRA:

1. Formulation of parameters of track quality entirely from data generated by automatic track geometry cars;
2. Identification of variables such as average speed and annual traffic load, which account for 80-90 percent of track deterioration as measured by TQIs;
3. Identification of the basic-maintenance tasks that optimize a given level of basic-maintenance expenditure;
4. Formulation of a track deterioration function that expresses the change in track quality as a function of track use and level of basic maintenance; and
5. Integration of the track deterioration model with an existing discretionary-maintenance planning methodology to form a combined model that can then be used as the basis for analytically planning both the basic and discretionary maintenance of track.

ACKNOWLEDGMENT

The cooperation of the Office of Freight Systems of the Federal Railroad Administration in supporting research in this important area and the graphics contributions of L. J. Zuclich and R. L. Dillon are greatly appreciated.

REFERENCES

1. Yearbook of Railroad Facts. Assn. of American

Railroads, Washington, DC, 1979.
2. S. D. Tayabji and M. Thompson. Considerations in the Analysis of Conventional Railway Track Support Systems. Transportation Engineering Journal, ASCE, Vol. 102, No. TE2, March 1977, pp. 280-291.
3. D. Staplin and others. Discretionary Track Maintenance Program: Volume 1—Methodology. Consolidated Rail Corp., Philadelphia, June 1978.
4. Acquisition and Use of Track Geometry Data in Maintenance-of-Way Planning. Federal Railroad Administration, U.S. Department of Transportation, Rept. FRA-ORD-75-27, March 1975.
5. Office of Research and Experiments Report on Question D177. Office of Research and Experiments, Utrecht, Netherlands, Rept. 7, Oct. 1977.
6. Parametric Study of Track Response. Federal Railroad Administration, U.S. Department of Transportation, Rept. FRA-ORD-77/75, Interim Rept., Dec. 1977.
7. G. P. Raymond and D. R. Williams. Repeated-Load Triaxial Tests on a Dolomite Ballast. Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT7, 1978, pp. 1013-1029.
8. Procedures for Analyzing the Economic Cost of Railroad Roadway for Pricing Purposes: Volume 1—Procedures. Tops On-Line Service, Inc., San Francisco, Jan. 1976. NTIS: PB 267 765.
9. G. P. Raymond. Design for Railroad Ballast and Subgrade Support. Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT1, 1978, pp. 45-60.
10. A Preliminary Description of Stresses in Railroad Track. Federal Railroad Administration, U.S. Department of Transportation, Rept. FRA-ORD-76-294, Interim Rept., Nov. 1976.

Publication of this paper sponsored by Task Force on Railway Maintenance.

PROFILE: Gradient Simulation for Rail Hump Classification Yards

William A. Stock, Masami Sakasita, Carola Elliott, and Peter J. Wong

Designers of rail hump yards traditionally execute a long, tedious manual process to optimally design hump grades and retarder placements. This design process entails checking the velocities and headways of a worst-case sequence of cars to ensure that proper values of these variables can be maintained on the gradient. The computer simulation model PROFILE automatically computes these quantities and thus frees the designer from tedious work and allows him or her to generate and study more design alternatives. The model uses the usual static (velocity-independent) rolling-resistance formulation of car rollability but includes the option of using velocity-dependent rolling resistance. User input requirements and program-generated output are described, and an example of the application of the model to a typical design problem is given.

In rail hump yards, classification is performed by rolling a cut of cars down a grade and switching the cars into various classification tracks. To perform switching properly, sufficient headway between cars must be created and maintained. The principal problems in the design of the hump profile and in the development of an effective speed-control scheme are to ensure that (a) the headway maintained in the switching area [e.g., 15.2 m (50 ft)] is sufficient to throw switches and prevent catch-up in retarders, (b) speed restrictions [e.g., 24.1 km/h (15 miles/h)] at switches and curves are observed, and (c) proper coupling occurs on the class tracks within specified speed limits [e.g., 3.2-9.7 km/h (2-6 miles/h)]. Controlling headway and speeds would not be difficult if all cars had identical characteristics and rolling resistances (or rollability) because the initial time separation established at the crest would result in a uniform and predictable headway between cars.

However, car rollability is not uniform; it varies with weather and type of car and changes during the rolling of a car. Nonetheless, the profile designer must ensure that a large percentage of the cars (e.g., 99.9 percent) are delivered to the bowl tracks in a manner that satisfies the above design constraints. Moreover, because car speed is directly translatable into hump throughput, it is desirable that the fastest car speeds meeting these constraints be used.

Achievement of these aims is usually approached by considering the hardest-rolling (slowest) and easiest-rolling (fastest) cars. Hump grades are usually designed to deliver the hardest-rolling car to the clear point at a specified speed [e.g., 6.4 km/h (4 miles/h)] or to a specified distance into the classification track [e.g., 152.4 m (500 ft)]. The sizing and placement of retarder sections are usually determined by examining a worst-case triplet of a design hardest-rolling car followed by a design easiest-rolling car followed by a design hardest-rolling car traveling to the last switch on the farthest outside track. The retarders are placed where the separation between the two lead cars becomes less than a specified value; the retarder slows down the second car to reestablish proper headway. The length (power) of the retarder is based on the amount of energy that must be removed from the second car in this worst-case situation (of course, railroad policies may require sufficient retarder power to stop any car). At the same time, caution must be exercised to ensure that the second (easiest-rolling) car is not slowed so much that the third (hardest-rolling) car catches it.

The purpose of the PROFILE model is to provide the yard designer with an iterative and interactive computer design tool to perform such an analysis and to ensure that the design constraints are satisfied. The need for some automation of the hump design procedure has long been recognized. The labor and hours involved in plotting velocity head diagrams and converting them to car velocity, integrating velocity of cars to obtain time-distance plots, and finally comparing time-distance plots of cars to obtain headway have severely restricted the number of design alternatives that the yard designer could consider. The PROFILE simulation model is intended to automate this process, and the automation also offers the designer the option of selecting a more advanced model of car rollability (over the usual static-rolling-resistance formulation), if desired. PROFILE does not automate the entire yard design process or replace the designer; it extends the abilities of the designer by permitting him or her to evaluate many more design alternatives in shorter time than is possible in the manual process.

The PROFILE model has been used to support the yard design efforts of the Boston and Maine Corporation (1), the Consolidated Rail Corporation (2), and the Union Pacific Railroad (3).

OVERVIEW OF THE MODEL

PROFILE is a one-track simulation; that is, the user selects one route from the crest to the bowl and simulates only that route in a run. With repeated runs, all routes to the bowl can be simulated, if necessary. The profile gradient along this route is represented as a series of track sections. All parameters are assumed to be constant within a given track section.

Only single-car cuts are modeled, although longer cuts can be approximated as a single car of unusual length.

Within each track section, each car is treated for the purpose of its dynamics as a point mass, the motion of which is assumed to be governed by the following differential equation:

$$d^2X/dt^2 = dV/dt = \alpha + \beta V \tag{1}$$

$$\alpha = g_e\,[\tan\theta - \mu - C - W - (S/L) - (R/L)] \tag{1a}$$

$$\beta = g_e\,(-\mu_V - W_V) \tag{1b}$$

$$g_e = [T/(T + I)]\,g \tag{1c}$$

where

- X = distance from an arbitrary origin (m);
- t = time (s);
- V = velocity of the car (m/s);
- α = sum of all static terms that contribute to the car's acceleration (m/s^2);
- β = sum of all velocity-dependent terms that contribute to the car's acceleration (s^{-1});
- g_e = effective acceleration of gravity used to account for energy stored in the rotating wheels of the car (m/s^2);
- g = acceleration of gravity (m/s^2);
- θ = angle of the grade below horizontal;
- $\tan\theta$ = grade (downgrades taken positive) (m/m);
- μ = static rolling resistance (N/N);
- C = curve resistance, if the track section is on a curve (N/N);
- W = wind resistance (N/N);
- S = velocity head lost in switch, if the track section is a switch (m);
- L = length of track section (m);
- R = velocity head extracted by retarder (if the track section is a retarder) (m);
- μ_V = velocity-dependent resistance coefficient (N/N per m/s);
- W_V = velocity-dependent wind resistance coefficient (N/N per m/s);
- T = weight of the car (kg); and
- I = additional weight of the car to account for the rotation of the wheels (kg).

Obviously, in any given track section, not all the terms will be applicable. For example, a conventional retarder and a switch would never be found in the same track section. The various parameters are assumed to be constant within each track section; whenever any parameters change, a new track section must be specified. This happens, for example, in specifying the beginning and end of a retarder. Specification of a new track section is also required whenever the grade changes. Vertical curves are approximated by a series of track sections of constant grade.

The solutions of the differential equations for $\beta \neq 0$ and taking $V = V_0$ and $X = X_0$ at $t = 0$ are

$$V = (\alpha/\beta) + [(\alpha/\beta) + V_0]\exp(\beta t) \tag{2}$$

and

$$X = X_0 - (\alpha/\beta)\,t - (1/\beta)\,[(\alpha/\beta) + V_0]\,[1 - \exp(\beta t)] \tag{3}$$

For $\beta = 0$ (i.e., only static rolling resistance), the solutions reduce to the well-known case of uniformly accelerated motion (for the above boundary conditions), as follows:

$$V = V_0 + \alpha t \tag{4}$$

and

$$X = X_0 + V_0 t + (1/2)\,\alpha t^2 \tag{5}$$

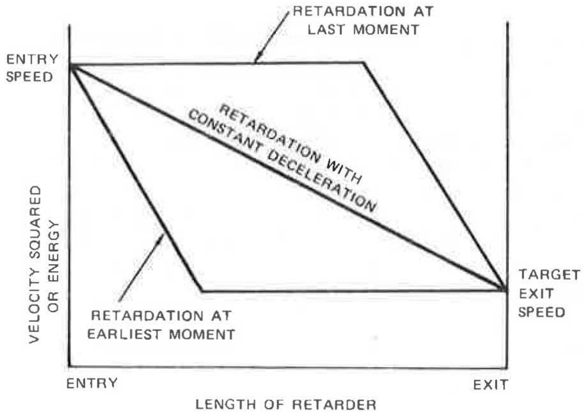
The $\beta = 0$ case is the usual static-rolling-resistance formulation, for which computational techniques based on energy relations are well developed. However, although these energy relations are easily applied to obtain velocity, integrating the velocities over a varying grade to obtain distance-time plots and hence headways between cars can become tedious. Even when a static-rolling-resistance formulation is being used, PROFILE has great utility as a quick means of calculation.

Although wind resistance would usually be handled by a V^2 term, in PROFILE only a V term is used. At the low speeds in a hump yard, the curvature of a V^2 relation should be sufficiently slight that it can be satisfactorily approximated by a linear term.

The retarders treated in the present version of PROFILE are the conventional clasp type, which are usually controlled by a process control computer. Currently, PROFILE does not consider the distributed types of retarders that offer quasicontinuous control through purely mechanical-hydraulic analog logic systems (as offered by certain European vendors). The conventional retarder system is quite complex: The process control computer controls both the overall amount of retardation and the detailed dynamics of car-retarder interactions while the car is within the retarder. Several algorithms are in use to decelerate the car within the retarder. They are all based on achieving a desired exit speed from the retarder. These algorithms can be roughly categorized into three types, as shown in Figure 1 and discussed below:

1. Retardation at the earliest moment—Retardation at the earliest moment is probably the most common algorithm for retarder control (4-6). The retarder closes as soon as the car enters; when the car reaches the exit velocity, either the retarder opens and the car rolls freely for the rest of the length of the retarder or the retarder opens and closes in an attempt to maintain

Figure 1. Retarder deceleration algorithms.



the car at approximately the desired exit speed. This scheme tends to restrict hump throughput because the car travels at minimum average speed for the length of the retarder. It also causes a disproportionate amount of retarder wear to occur near the front.

2. Retardation at the last moment (4)—The algorithm for last-moment retardation is based on a prediction of the rollability of the car and the power of the retarder. The retarder initially remains open when the car enters it. By using the predicted parameters, the retarder is then closed just at the time expected to produce deceleration of the car to the desired exit velocity. This scheme generally permits a high throughput because the car moves at maximum speed throughout the retarder. This algorithm, however, lacks a safety margin for cases in which the car rolls faster than predicted because of errors in predicting rollability, grease on the wheels or rails, or the like. This algorithm also causes a disproportionate amount of retarder wear to occur near the rear of the retarder.

3. Retardation with constant deceleration—Under the constant-deceleration algorithm, the retarder is commanded either to open and close several times (7) or to exert a constant retardation force (5); in either case, the aim is to achieve the desired exit speed with approximately constant deceleration. Some modern commercial retarder systems achieve this ideal at least approximately (8). This scheme maintains better throughput than algorithm 1 and maintains a safety reserve of retarder power that is lacking in algorithm 2. It also causes the retarder to wear approximately uniformly throughout.

In the PROFILE model, the third type of deceleration scheme, constant deceleration, is assumed to apply. Under constant deceleration, energy (i.e., velocity head) is extracted at a uniform rate during the car's transit of the retarder, and the total amount of velocity head extracted within the retarder, when divided by the retarder length, acts simply as an additional resistance term—hence, its appearance in Equation 2.

PROGRAM DESCRIPTION

PROFILE is a time-step simulation written in ANSI standard FORTRAN. Events are assumed to occur either at integral multiples of a predetermined time step Δt or within the time step for certain easily calculated events (such as the entry of a car into a new track section). The time-step method has been selected because of the ease it affords in the calculation of transcendental solutions to differential equations.

The simulation starts by humping the first car at simulation clock time zero. From the length of the cars involved and the hump speed, the hump time for the second car is computed and stored until the simulation clock is equal to that hump time. At the calculated hump time, the second car is humped and put into the system. The hump time for each car is so computed until all cars that the user wishes to put into the system are humped.

Once a car has been humped, movement of cars along the track is accomplished by advancing the simulation clock in increments of Δt. At each time step, the differential Equation 1 is solved for the instantaneous velocities and the distances of cars along the track. Each time a car enters a new track section, the program solves an initial-value problem based on the general solution to the differential equation and the specified configurations of the new track segment. These coefficients are used in subsequent calculations for this car on the track at steps of Δt until the car leaves the track section.

At each time step, the coupler-to-coupler headways between the cars in the system are checked to maintain a safe operating distance between the cars and to avoid mis-switching, catch-up in retarders, and collisions. If headway is insufficient, the program writes a warning message to the output file. If a collision occurs or if a car stalls, the program stops and writes a message to the output file. These messages show the simulation clock time when the catch-up occurred, the distance along the track for each car, and the velocities of the cars at that time. The user can then analyze the output and change retarder placements, the length of the retarder, or any other parameter and start a new computer iteration.

Data on each car are collected at each print interval as specified by the user. For each car, the simulation clock time, instantaneous velocity, velocity head, distance from the hump crest, and distance and time headways from the preceding car are written to and stored in a print buffer. Data in the buffer are written to the output file whenever the simulation stops. If no collision or stall occurs, the simulation stops when the last car has come to the end of the last track section.

Figures 2-4 show sample partial outputs (the program is calibrated in U.S. customary units of measurement).

DESCRIPTION OF INPUT

The first input variables are general: the time step Δt, the hump speed, the data print interval, switches controlling the printing of tables and plots, and the printer width (in characters). To model the occurrence of events accurately, the time step chosen should be sufficiently small but not so small as to cause an inordinate increase in running time (1 s is usually satisfactory). Data output frequency is controlled by the data print interval variable, which should be chosen in integral multiples of the time step but should never be less than the time step.

Next, the following data for the track sections are specified (in U.S. customary units):

1. Length of track section (ft);
2. Grade of track (percent);
3. Rolling resistance, static, easy roller (lbf/ton·f);
4. Rolling resistance, static, hard roller (lbf/ton·f);

5. Rolling resistance, velocity, easy (lbf/ton·f per ft/s);
6. Rolling resistance, velocity, hard (lbf/ton·f per ft/s);
7. If the section is a switch, switch loss, in velocity head (ft);
8. If the section is a retarder, amount of retardation to be given easy-rolling car, in velocity head (ft);
9. If the section is a retarder, amount of retardation to be given hard-rolling car, in velocity head (ft);
10. If the section is a retarder, maximum retardation of the retarder, in velocity head (ft); and
11. Track section alphanumeric identification.

The static and velocity resistances can be specified separately for each track section for the two types of car—easy or hard rolling. Specifying rolling resistances in this manner allows them to vary along the simulated track. If the track segment is a switch or retarder section, additional parameters are required as shown.

Additional data for the cars constitute the final set of information specified to the program. First, the type of car must be specified (easy or hard rolling). Then the car length, in feet; the weight of the car, in tons; and an equivalent rotational weight for the wheels, in tons, must be given. Each car is associated with static (lbf/ton·f) and velocity-dependent (lbf/ton·f per ft/s) wind-resistance terms. These values may vary depending on the type of car (box car, flat car, gondola, etc.).

DESCRIPTION OF OUTPUT

The output from PROFILE consists of four parts. The first part is an "echo-back" of the input data (Figure 2), simply a listing of the user's input given for documentation and verification. The second part, which immediately follows the echo-back of car data, lists any special events that might have occurred during the simulation, such as a catch-up of two cars within a retarder, a collision between two cars, or a car stalling. Note that, if no special events occurred, this portion of the output is omitted.

The third part is the numerical output from the simulation proper. This consists of a series of tables, one table for each car. Figure 3 shows an example of a portion of such a table. Each line in a table gives a number of variables that define the status of that particular car at a point in time. The lines are generally printed at uniform increments of simulated time, although whenever a car enters a new track section an additional line is printed. The print increment is specified by the user and is usually on the order of 1 s.

The fourth output section gives optional line printer plots of selected variables. These plots, which include relevant annotation, consist of (a) yard profile versus

Figure 2. Echo-back and collision information for trial run 2 of Yermo Yard.

SRI HUMP PROFILE SIMULATION - YERMO NO. 8 FILE - TRIAL RUN 2 - ELIMINATE MASTER RETARDER

SIMULATION TIME STEP, DELTA T, SEC	1.0000
HUMP SPEED, MILES PER HOUR	2.5000
DATA PRINT INTERVAL, SEC	1.00
TABLE SWITCH	1
PLOT SWITCH	1
PRINTER WIDTH (CHARACTERS)	132

T R A C K   D A T A

TRK SEC NO.	LENG (FT)	CUM LENG (FT)	GRADE (PCT)	RESISTANCES ROLLING STATIC (LB/TON) EASY	RESISTANCES ROLLING STATIC (LB/TON) HARD	RESISTANCES ROLLING VELOCITY (LB/TON)/(FT/SEC) EASY	RESISTANCES ROLLING VELOCITY (LB/TON)/(FT/SEC) HARD	RESISTANCES HORIZ. CURVE (LB/T)	SWITCH LOSS (FT OF VELOC. HEAD)	RETARDATION (FT. OF VEL. HEAD) CAR 1	RETARDATION (FT. OF VEL. HEAD) CAR 2	RETARDATION (FT. OF VEL. HEAD) CAR 3	MAX. RETARDATION (FT OF VELOC. HEAD)	D E S C R I P T I O N
1	50.0	0.0	3.00	4.00	18.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	CREST TO EVC
2	71.0	50.0	4.23	4.00	18.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	EVC TO FORMER M. RET.
3	72.0	121.0	3.99	4.00	18.00	-0.00	-0.00	-0.00	-0.00	0.00	0.00	0.00	6.72	FORMER MASTER RET.
4	24.0	193.0	1.44	4.00	18.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	FORMER M. RET. TO KING SW.
5	1.0	217.0	1.44	4.00	18.00	-0.00	-0.00	-0.00	.06	-0.00	-0.00	-0.00	-0.00	KING SW
6	25.0	218.0	.50	4.00	18.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	KING SW TO LAP
7	1.0	243.0	.50	4.00	18.00	-0.00	-0.00	-0.00	.06	-0.00	-0.00	-0.00	-0.00	LAP SW
8	100.0	244.0	.50	4.00	18.00	-0.00	-0.00	11.30	-0.00	-0.00	-0.00	-0.00	-0.00	LAP SW TO PT
9	95.0	344.0	.50	4.00	18.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	PT TO GR. RET.
10	100.0	439.0	1.20	4.00	18.00	-0.00	-0.00	-0.00	-0.00	0.00	5.24	0.00	6.72	GR. RET.
11	17.0	539.0	.10	2.00	10.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	GR TO LAP 2
12	1.0	556.0	.10	2.00	10.00	-0.00	-0.00	-0.00	.06	-0.00	-0.00	-0.00	-0.00	LAP 2
13	70.0	557.0	.10	2.00	10.00	-0.00	-0.00	5.25	-0.00	-0.00	-0.00	-0.00	-0.00	LAP 2 TO HF 2
14	28.0	627.0	.10	2.00	10.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	HF 2 TO SW 3
15	1.0	655.0	.10	2.00	10.00	-0.00	-0.00	-0.00	.06	-0.00	-0.00	-0.00	-0.00	SW 3
16	70.0	656.0	.10	2.00	10.00	-0.00	-0.00	5.25	-0.00	-0.00	-0.00	-0.00	-0.00	SW 3 TO HF 3
17	30.0	726.0	.10	2.00	10.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	HF 3 TO SW 4
18	1.0	756.0	.10	2.00	10.00	-0.00	-0.00	-0.00	.06	-0.00	-0.00	-0.00	-0.00	SW 4
19	70.0	757.0	.10	2.00	10.00	-0.00	-0.00	5.25	-0.00	-0.00	-0.00	-0.00	-0.00	SW 4 TO HF 4
20	68.0	827.0	.10	2.00	10.00	-0.00	-0.00	12.25	-0.00	-0.00	-0.00	-0.00	-0.00	HF 4 TO CLEAR
21	145.0	895.0	.10	2.00	10.00	-0.00	-0.00	12.25	-0.00	-0.00	-0.00	-0.00	-0.00	CLEAR TO POT
22	15.0	1040.0	.10	2.00	10.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	POT TO PTT
23	300.0	1055.0	.08	2.00	10.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	-0.00	PTT TO END

C A R   D A T A

TYPE OF ROLLER, 1 = EASY, 2 = HARD

CAR NO.	TYPE ROLLER	CAR LENGTH (FT)	WEIGHT OF CAR (TONS)	EXTRA WEIGHT WHEEL ROTATION (TONS)	WIND RESIS STAT (LB/T)	WIND RESIS VELOC (LB/T)/(FPS)
1	2	60.00	64.00	1.00	-0.00	-0.00
2	1	60.00	135.00	1.00	-0.00	-0.00
3	2	60.00	64.00	1.00	-0.00	-0.00

A COLLISION OCCURRED AT TIME 93.82 SEC. BETWEEN
CAR 1 - VEL = 3.02 MPH, DIST = 1129.01 FT., TIME ON TRACK = 93.82 SEC.
CAR 2 - VEL = 6.00 MPH, DIST = 1069.01 FT., TIME ON TRACK = 77.45 SEC.

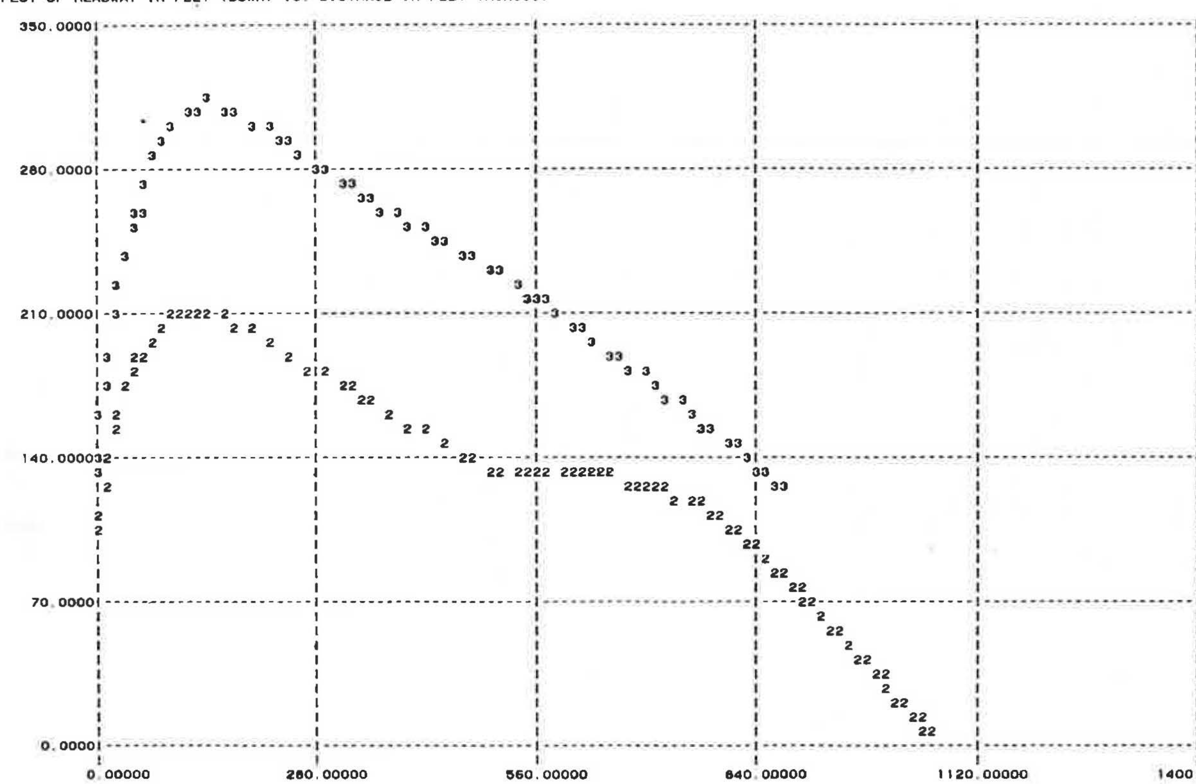
Figure 3. Example of car history table: partial output of car 2 (easy roller) for trial run 2 of Yermo Yard.

CAR NO. 2

CAR TRAVEL TIME (SEC)	SYSTEM TIME (SEC)	DISTANCE ALONG TRACK (FT)	DISTANCE HEADWAY BETWEEN PREC CAR (FT)	TIME HEADWAY BETWEEN PREC CAR (SEC)	INSTAN-TANEOUS VELOCITY (FT/SEC)	INSTAN-TANEOUS VELOCITY (MPH)	VELOCITY HEAD (FT)	TRACK SECTION NUMBER	TRACK SECTION DESCRIPTION
0.000	16.364	0.000	102.770	7.382	3.667	2.500	.210	0/ 1	****TRACK SECTION BOUNDARY****
.636	17.000	2.515	111.691	7.772	4.236	2.888	.281	1	CREST TO EVC
1.636	18.000	7.198	125.776	8.328	5.131	3.499	.412	1	CREST TO EVC
2.636	19.000	12.777	139.547	8.823	6.026	4.109	.568	1	CREST TO EVC
3.636	20.000	19.250	152.447	9.266	6.921	4.719	.749	1	CREST TO EVC
4.636	21.000	26.619	164.261	9.663	7.816	5.329	.956	1	CREST TO EVC
5.636	22.000	34.883	174.850	10.022	8.711	5.939	1.187	1	CREST TO EVC
6.636	23.000	44.041	184.238	10.347	9.606	6.550	1.443	1	CREST TO EVC
7.240	23.603	50.000	189.322	10.530	10.146	6.918	1.610	1/ 2	****TRACK SECTION BOUNDARY****
7.636	24.000	54.126	192.394	10.642	10.657	7.266	1.777	2	EVC TO FORMER M. RET.
8.636	25.000	65.427	199.028	10.892	11.945	8.145	2.232	2	EVC TO FORMER M. RET.
9.636	26.000	78.016	204.067	11.097	13.234	9.023	2.740	2	EVC TO FORMER M. RET.
10.636	27.000	91.694	207.583	11.263	14.522	9.901	3.299	2	EVC TO FORMER M. RET.
11.636	28.000	107.060	209.683	11.395	15.810	10.779	3.910	2	EVC TO FORMER M. RET.
12.489	28.852	121.000	210.356	11.484	16.908	11.528	4.472	2/ 3	****TRACK SECTION BOUNDARY****
12.636	29.000	123.513	210.369	11.498	17.087	11.651	4.567	3	FORMER MASTER RET.
13.636	30.000	141.206	209.688	11.574	18.299	12.477	5.238	3	FORMER MASTER RET.
14.636	31.000	160.111	207.669	11.599	19.510	13.302	5.955	3	FORMER MASTER RET.
15.636	32.000	180.227	204.324	11.557	20.722	14.128	6.717	3	FORMER MASTER RET.
16.242	32.806	193.000	201.725	11.494	21.456	14.629	7.201	3/ 4	****TRACK SECTION BOUNDARY****
16.636	33.000	201.491	199.877	11.439	21.612	14.735	7.306	4	FORMER M. RET. TO KING SW.
17.349	33.713	217.000	196.445	11.323	21.895	14.928	7.499	4/ 5	****TRACK SECTION BOUNDARY****
17.395	33.759	218.000	196.189	11.315	21.825	14.881	7.451	5/ 6	****TRACK SECTION BOUNDARY****
17.636	34.000	223.269	195.010	11.272	21.848	14.896	7.467	6	KING SW TO LAP
18.538	34.901	243.000	190.603	11.096	21.935	14.955	7.526	6/ 7	****TRACK SECTION BOUNDARY****
18.583	34.947	244.000	190.381	11.086	21.851	14.899	7.469	7/ 8	****TRACK SECTION BOUNDARY****
18.636	35.000	245.158	190.127	11.075	21.847	14.896	7.466	8	LAP SW TO PT
19.636	36.000	266.962	185.424	10.859	21.762	14.838	7.408	8	LAP SW TO PT
20.636	37.000	288.682	180.901	10.622	21.677	14.780	7.351	8	LAP SW TO PT
21.636	38.000	310.317	176.535	10.373	21.593	14.722	7.293	8	LAP SW TO PT
22.636	39.000	331.868	172.104	10.118	21.508	14.665	7.236	8	LAP SW TO PT
23.201	39.565	344.000	169.520	9.971	21.460	14.632	7.204	8/ 9	****TRACK SECTION BOUNDARY****
23.636	40.000	353.351	167.483	9.855	21.502	14.660	7.232	9	PT TO GR. RET.
24.636	41.000	374.900	162.586	9.576	21.598	14.726	7.297	9	PT TO GR. RET.
25.636	42.000	396.546	157.381	9.286	21.694	14.791	7.362	9	PT TO GR. RET.
26.636	43.000	418.288	151.873	8.999	21.790	14.857	7.427	9	PT TO GR. RET.
27.585	43.949	439.000	146.418	8.731	21.881	14.919	7.489	9/10	****TRACK SECTION BOUNDARY****
27.636	44.000	440.124	146.116	8.717	21.811	14.871	7.442	10	GR. RET.
28.636	45.000	461.257	140.879	8.484	20.456	13.947	6.546	10	GR. RET.
29.636	46.000	481.035	136.734	8.337	19.100	13.023	5.707	10	GR. RET.
30.636	47.000	499.458	133.733	8.266	17.745	12.099	4.926	10	GR. RET.
31.636	48.000	516.525	131.878	8.257	16.390	11.175	4.202	10	GR. RET.
32.636	49.000	532.237	131.168	8.317	15.035	10.251	3.536	10	GR. RET.
33.096	49.459	539.000	131.229	8.368	14.412	9.827	3.249	10/11	****TRACK SECTION BOUNDARY****
33.636	50.000	546.793	131.433	8.435	14.412	9.827	3.249	11	GR TO LAP 2
34.275	50.639	556.000	131.627	8.512	14.412	9.827	3.249	11/12	****TRACK SECTION BOUNDARY****
34.345	50.709	557.000	131.649	8.520	14.279	9.736	3.189	12/13	****TRACK SECTION BOUNDARY****
34.636	51.000	561.158	131.760	8.556	14.255	9.719	3.179	13	LAP 2 TO HF 2

Figure 4. Distance headway versus distance for trial run 2 of Yermo Yard.

PLOT OF HEADWAY IN FEET (DOWN) VS. DISTANCE IN FEET (ACROSS)



PLOT LEGEND -

HEADWAY BT. CARS	PLOTTED NO.
1 AND 2	2
2 AND 3	3

distance, (b) speeds of all cars versus distance, and (c) distance headways between all cars versus distance (Figure 4).

APPLICATION OF THE MODEL

The sample application problem described in this section is based on a modified specification for the Union Pacific Railroad's Yermo Yard in southern California. The hump profile design requires several levels of decision making on cost- and performance-related matters. The considerations on cost and performance would be reflected in the retarder types to be used, hump-crest height, humping speed, impact speed, and number of mis-switched cars. After having determined the type of retarder and retarder configuration to be adopted, the designer must iteratively examine both the horizontal and vertical design to arrive at a final design that satisfies the specified goal.

The application problem discussed here is only one stage of the process of hump profile design in which a given profile design is evaluated and modified to a better design through iterations of PROFILE runs.

The design, as used in trial run 1 (not shown in the figures) in this example, has a master retarder of 28.3 m (93 ft) and three group retarders of 30.5 m (100 ft). Each group retarder leads to 10 classification tracks. The distance between the hump crest and the tangent point of the outermost track is 323.4 m (1061 ft).

The runs for this design were based on the simulation of a conventional hard/easy/hard-rolling triplet of cars. A worst-case condition was assumed: Since the easy-rolling car is going to a nearly full-class track, it must be retarded to a low target speed by the tangent point [9.6 km/h (6 miles/h)]; meanwhile, since the hard-rolling car must penetrate as far as possible an adjacent empty-class track, its retardation is minimal.

The objective of the study was to test the feasibility of the design by examining the following design requirements:

1. The hump speed is at least 4.0 km/h (2.5 miles/h), 3.67 cars/min.
2. The hard roller must not stall before the tangent point.
3. The maximum speed of the easy roller at the tangent point is 9.6 km/h (6 miles/h).
4. The maximum speed of a car in the switch segments is 24 km/h (15 miles/h).
5. The coupler-to-coupler headway is at least 15.2 m (50 ft) at each switch.
6. There is never more than one car in the same retarder at any time.
7. No catch-ups should occur before the clearance point of each track.

The major assumptions used in the design process were the following:

1. Only static rolling resistances apply.
2. The hard roller has a rolling resistance of 9 N/kN (18 lbf/ton·f) between the hump crest and the exit from the group retarders and 9 N/kN (10 lbf/ton·f) thereafter.
3. The easy roller has a rolling resistance of 2 N/kN (4 lbf/ton·f) between the hump crest and the exit from the group retarder and 1 N/kN (2 lbf/ton·f) thereafter.
4. The velocity head loss attributable to each switch is 0.018 m (0.06 ft) when the car travels along the curved track and is assumed to be zero if a car travels on the straight track. This value is constant for all turnout numbers.
5. The velocity head loss attributable to a curved section of track is 0.012 m (0.04 ft) per degree of deflection angle.
6. The average car length is 18.3 m (60 ft).
7. The average car weight is 58 Mg (64 tons) for the hard roller and 122 Mg (135 tons) for the easy roller.
8. The extra weight of the car attributable to wheel rotation is 0.91 Mg (1 ton).
9. The wind resistance is zero.

A general interactive and iterative design procedure was used here to select an example design. The steps in this procedure are the following:

1. Select the configuration and type of retarder and the method of retardation.
2. Determine the car-speed constraints at the tangent point and at other points along the track.
3. Design a trial horizontal layout.
4. Determine the hump height from steps 2 and 3.
5. Select the trial grades along the track.
6. Run PROFILE.
7. Examine the output. If the result is satisfactory, go to step 8. If the result shows speed violations, go back to step 3. If the result contains catch-up problems, go first to step 5. If the catch-up problem cannot be solved by changing grades, go to step 3.
8. Determine whether any segment, especially the retarder segment, is excessively long; if so, go to step 3. Otherwise, the design is complete.

It should be noted that other procedures not shown here have been developed to enable the PROFILE user to select a hump speed and a retarder control policy.

The example discussed here illustrates one step of the interactive and iterative design procedure presented above. The objective in trial run 2, the partial output of which is shown in Figures 2-4, was to try to eliminate the master retarder. This change necessitated shortening the distance between the hump crest and the first switch by 6.4 m (21 ft), which shortened the distance to the tangent point to 317.0 m (1040 ft). A comparison between the collision-related output for trial run 1 (not shown) and the same information for trial run 2 (Figure 2) revealed that the collision point decreased from 398.1 to 334.0 m (1306-1099 ft) from the hump crest. Since the latter value is still well past the clearance point (in fact, past the tangent point), the design of trial run 2 satisfies the design requirements. Examination of other performance measures output by the model, as shown partially in Figures 2-4, reveals that all other design requirements are also met by the design of trial run 2. Under the assumptions used in this example, the design changes effected between trial runs 1 and 2 demonstrate a considerable cost reduction and point up the advantage of having the PROFILE model available to try such "what if" experiments.

Figure 3 shows a part of the output for car 2 (the easy roller). All of the necessary data related to the movements of car 2 are included in this table.

From the plot of speeds of the cars as a function of distance (not shown) or data such as those in Figure 3, it has been determined that the easy roller in trial run 2 attains a maximum speed of slightly less than 24 km/h (15 miles/h). This satisfies the maximum-speed constraint in the switching area. It can also be verified that the easy roller satisfies the 9.6-km/h (6-mile/h) speed constraint at the tangent point and that the unretarded hard roller satisfies both speed constraints.

Figure 4 shows a plot of distance headway between successive cars. The number 2 indicates the headway between cars 1 and 2, and 3 indicates the headway between cars 2 and 3. Figures 3 and 4 show that sufficient headway exists between cars to detect individual cars and to throw the switch in all switch segments.

FURTHER WORK AND CONCLUSIONS

Further work is in progress to enhance the interactive capability of the PROFILE program. Specifically, simplifying the user input procedures and increasing the amount of graphical output are being considered. In addition, more work is required to characterize and quantify the nature of car rollability. Freight-car rolling behavior, which is essentially an input to PROFILE, is a critical determinant of the final profile design.

This paper has shown that PROFILE can be used to eliminate the tedious manual process of evaluating hump profile designs by using scale drawings. In addition, PROFILE gives a precise prediction of catch-up problems between cars. The program allows the yard designer to evaluate many more design alternatives than it was previously possible to evaluate, thus ensuring production of the most cost-effective design.

ACKNOWLEDGMENT

The work reported in this paper was performed under a project for the Transportation Systems Center in Cambridge, Massachusetts, to improve the state of the art in railroad classification yard design. Technical monitor John Hopkins supervised the work. The Office of Freight Systems of the Federal Railroad Administration, under the program management of William F. Cracker, Jr., sponsored the work.

We acknowledge the technical assistance and support of J. Wetzel of the Consolidated Rail Corporation, M. J. Anderson of the Union Pacific Railroad, B. Gallacher of the Southern Pacific Transportation Company, and Vinay Mudholkar of the Boston and Maine Corporation.

REFERENCES

1. M. Sakasita, M. A. Hackworth, P. J. Wong, V. V. Mudholkar, and D. B. Koretz. East Deerfield Yard Rehabilitation: A Case Study. Proc., Classification Yard Technology Workshop, Chicago, Oct. 1979.
2. C. V. Elliott, M. Sakasita, W. A. Stock, P. J. Wong, and J. Wetzel. Elkhart Yard Rehabilitation: A Case Study. Proc., Classification Yard Technology Workshop, Chicago, Oct. 1979.
3. M. J. Anderson. Overview of Key Design Procedures for Union Pacific's Yermo Yard Proposal. Proc., Classification Yard Technology Workshop, Chicago, Oct. 1979.
4. H. Konig. Control Algorithms for Rail Retarders and Closing-Up Devices in Marshalling Yards. Monthly Bull., International Railway Congress Assn., Vol. 5, No. 12, Dec. 1969.
5. R. J. Berti. Automatic Control Means for Retarders. U.S. Patent Office, U.S. Patent 3 283 146, Nov. 1, 1966.
6. R. J. Budway and G. F. McGlumphy. Retarder Control Systems for Automatic Railroad Classification Yards. U.S. Patent Office, U.S. Patent 3 946 973, March 30, 1976.
7. P. J. Wong and R. S. Ratner. Hump Yard Retarder Control System. U.S. Patent Office, U.S. Patent 3 745 334, July 10, 1973.
8. VR-3A Speed Control System for Seaboard Coast Line Railroad Company Rice Yard: Operation and Maintenance. Union Switch and Signal Division, Westinghouse Air Brake Co., Pittsburgh, Service Manual 6084, Aug. 1977.

Publication of this paper sponsored by Task Force on Railroad Operations.

Conflicts Between Urban Areas and Railroads: A Status Report

Richard G. McGinnis

The development of conflicts between urban areas and railroads in the United States is examined, and the nature and magnitude of the current problems and present and past efforts to resolve them are described. Many American cities developed primarily as a result of the railroads, but changes in urban activities and transportation operations have altered somewhat the relation between the cities and railroads. Continuing expansion of urbanized areas and increases in vehicle travel have intensified the conflict. Cities have reacted by pushing for elimination of railroad-highway grade crossings and, in some cases, for consolidation, relocation, and/or removal of railroad tracks from the center city. Many city planners see the railroads as a hindrance to rejuvenation efforts. In some cities, underutilized railroad properties are in strategic locations that could be important in urban redevelopment plans. High-volume rail lines that pass through congested downtown areas can cause massive traffic jams and delays unless crossings are grade separated. Railroad-highway grade crossings pose safety problems to the motorist and restrict mobility, which is particularly important for emergency vehicles. In addition, the slow train speeds mandated by local municipalities, frequent grade crossings, and large numbers of trespassers are not compatible with efficient railroad operation. But new rail routes are difficult to locate and expensive to build, and there are many implementation problems involved in other, less expensive solutions, such as consolidation or abandonment.

Conflicts between U.S. railroads and urban communities have existed, to varying degrees, ever since railroad operations began in 1830. Initially, most of the concerns about urban railroads had to do with safety. Safety problems included dangers associated with grade crossings, runaway trains, and derailments. However, since train speeds through towns were relatively slow and vehicle traffic crossing tracks was of low volume, the safety of rail operations in urban areas was of relatively minor importance during the 19th century.

Although concerns about conflicts between urban areas and railroads were articulated frequently during the early development of the railroads, most cities realized the great economic advantages inherent in railroad development. Thus, community leaders often competed fiercely to entice railroads to locate in their communities rather than in neighboring towns. Towns located at railroad terminals and transshipment points grew rapidly in population.

Initially, inefficient means of overland transport encouraged most new urban development to occur close to the rail lines and to form linear and radial types of development patterns. The desirability of being close to railroads began to decrease as train operations increased in speed and frequency and highway transportation improved. By the early 1900s, conflicts between motor vehicles and trains at grade crossings were becoming a major safety problem. Rail lines were physical and sometimes psychological barriers that separated one type of a neighborhood or land use from another type, and residents were becoming less tolerant of the adverse environmental impacts of the trains.

By 1920, the railroads' dominance of the transportation market was beginning to wane. The near-monopoly enjoyed by the railroads during their first 90 years of operation was being threatened by the truck and the automobile while government was imposing tighter controls on railroad operations. Cities were reacting to their growing conflict with the railroads by planning for grade separations, studying possibilities for railroad relocations, and implementing ordinances to control train operations through towns.

Between 1929 and 1970, railroad consolidations and abandonments resulted in a 17 percent decrease in kilometers of railroad lines, to 332 685 km (206 265 miles). More efficient freight operations, because of longer trains with higher-capacity freight cars, permitted a 30 percent reduction in train kilometers, although ton kilometers of freight increased by 71 percent (1).

Much of the new urban development that has taken place since 1920 has occurred at population densities lower than those that prevailed before 1920. The greater flexibility provided by automobiles allowed residents to move farther away from established employment centers and rail transportation facilities. Expansion of the urban street system and concomitant increases in volumes of motor vehicle traffic resulted in an intensification of the problem of highway-railroad grade crossings.

CURRENT PROBLEMS

The nature and magnitude of conflicts between railroads and urban areas vary widely among communities but can usually be classified into one of five categories: safety problems, mobility constraints, environmental problems, land use conflicts, and railroad operational problems.

Safety Problems

Concern about the safety of railroad operations in urban areas is probably the most common problem aired in discussions of railroad relocation projects. Most safety problems arise from conflicts involving railroad-highway grade crossings, fears about train accidents involving rail cars that are carrying hazardous materials, or dangers to pedestrians and trespassers.

Concern for safety at railroad-highway grade crossings has resulted in the construction of grade separations or installation of gates and/or flashing lights at some of the more dangerous crossings. However, of the 95 102 grade crossings in urban areas, 65 228 still have only passive warning devices, such as crossbucks and stop signs (2).

Current accident statistics do not provide for breakdowns between urban and rural accidents. It has been estimated, however, that 60 percent of accidents at highway-railroad grade crossings occur in urban areas (3). Estimates for 1977 (4) are that 7375 accidents occurred at urban railroad-highway grade crossings, including 375 fatalities and 2775 injuries. If we use values established by the National Highway Traffic Safety Administration for the societal costs of fatalities and injuries (5) and update these values to 1978 dollars by using a 6 percent annual inflation rate, the total economic cost to society of railroad-highway grade-crossing accidents in urban areas is estimated to be about $150 million/year.

Seventy percent of the ton kilometers of transportation of hazardous materials in the United States is by rail. During 1975 and 1976, railroads were responsible for only 2 fatalities involving hazardous materials and trucks for 43 (6). Of the 1977 fatalities that involved hazardous materials, 4 were caused by railroads and 30 by trucks (7). Because of the safety advantage that railroads have over trucks in the movement of hazardous materials, it is likely that railroads will continue to transport major quantities of hazardous materials in the future.

Hazardous materials, which include toxic materials, explosives, flammable products, corrosive substances, and radioactive materials, present potentially catastrophic dangers to the urban areas through which they must pass during transport. Because the main lines of many railroads pass through urbanized areas, it is very difficult, if not impossible, to route these materials around cities. Extensive precautions are taken by the railroads when they move hazardous materials; however, the tremendous volumes of these materials that are being moved by rail increase the probability that some accidents will occur, although this probability is significantly lower than it would be if movement were by truck.

During 1977, 525 pedestrians and trespassers were killed on railroad facilities. Trains and railroad yards have always been an attraction, especially to children. Seven-eighths of the people killed were illegally trespassing on railroad property (7). Applying the values used in the grade-crossing analysis for societal costs of fatalities, a societal cost of $185 million/year can be placed on railroad-pedestrian casualties.

Mobility Constraints

Non-grade-separated railroads in urban areas act as barriers to highway and pedestrian transportation, and delays are experienced at grade crossings whenever a train occupies the intersection. Furthermore, street networks are often distorted so that drivers must travel a circuitous route to get from one side of the tracks to the other. Travel delays are especially detrimental to emergency vehicles, and some communities have had to build additional police stations and firehouses to ensure that essential services are available to both sides of the track at all times. The annual cost of time delays and additional operating costs attributed to urban grade crossings in the United States is estimated to be about $1 billion in 1978 (8).

Railroads have historically created psychological barriers in urban areas, often separating neighborhoods of differing ethnic or socioeconomic characteristics, which sometimes leads to designations of certain areas as the "right" side or "wrong" side of the tracks. Some city planners also feel that railroads that limit access to a central business district (CBD) are preventing, or at least inhibiting, revitalization of the center city. Few data exist to show whether or not removal of a railroad would eliminate the psychological barriers that have developed over long periods of time. However, most studies have concluded that railroad relocation is not a panacea for revitalizing urban areas.

Environmental Problems

Most railroads were designed and constructed at a time when function rather than environmental compatibility was the primary design criterion. This philosophy carries over today in freight car design, right-of-way maintenance procedures, and the architecture of many railroad facilities and generally has a negative effect on the aesthetics of bordering neighborhoods.

In addition to negative impacts on the visual quality of an area, railroads produce noise, vibrations, air pollution, and, in some areas, water pollution.

The magnitude of these environmental problems increases with the density of train operations and the population density of the areas adjacent to the railroad. Noise problems emanate from such factors as train horns, locomotive noises, the squeal of brakes and steel wheels on sharp curves, grade-crossing warning bells, and classification yards. Noise problems are worse in the vicinity of grade crossings because of the legal requirement in most states that trains sound their horns before each grade crossing as a warning to motorists.

The environmental intrusion of the railroad is reflected in the value of land ajacent to the right-of-way. The negative impact of railroad operations on property values is greatest for residential areas; thus, decreases in property values close to railroads are a measure of the "cost" of the environmental degradation caused by the presence of the railroad.

Land Use Conflicts

Railroad facilities, particularly in urban areas, often restrict higher-value land use. Many cities have CBDs that contain sizable tracts of land owned by railroads, and in some cases these facilities are abandoned, out-of-service, or underutilized by the railroads. Freeing this land for other land uses could be an important first step in revitalizing the CBD.

Even if the railroad facilities are currently being fully used, restructuring of rail operations, including relocation of some facilities, may prove to be cost-beneficial for community development reasons. Over the years, changes in railroad operations may have obviated the need to have certain operations in downtown business districts.

Railroad Operational Problems

Railroads are not able to operate as efficiently in urban areas as they do in less congested rural areas. Most operational problems associated with urban operations can be attributed to two causes: slow running speeds and large numbers of grade crossings.

Grade-crossing accidents, in addition to killing and injuring many people, inflict time and money costs on the railroads. These costs result principally from disruptions in operations, filing of accident reports, damages to equipment, and liability suits. In addition, most states require the railroads to maintain grade-crossing warning devices and crossing surfaces.

Many local municipalities have ordinances that set maximum running speeds for trains through urbanized areas. In some cities, the tracks actually run right down the middle of busy downtown streets and trains are restricted to maximum speeds of 8 km/h (5 miles/h). These low speeds, while necessary for safety reasons, delay train movements, increase labor costs, and decrease equipment utilization, all of which leads to inefficient operation. In 1970, the annual cost to the railroads associated with low-speed operation in urban areas was estimated to be approximately $75 million-$100 million (8).

PRESENT AND PAST EFFORTS TO RESOLVE CONFLICTS

Conflicts between railroads and urban areas can be ameliorated in either of two ways. The most common way, and usually the less expensive one, is to modify the railroad-urban area interface so that the conflict is either eliminated or minimized. Warning devices and/or grade separations at grade crossings, elevation or depression of rail lines, and installation of buffer zones or barriers along rail lines are examples of methods that can be used to reduce conflicts between urban areas and railroads. A second approach is to remove the railroad by either abandoning it or relocating it. These solutions are usually considerably more expensive than the above method, but they do entirely eliminate the conflict.

Grade-Crossing Protection and Elimination Programs

Background

From 1920 to 1930, the railroads carried out an extensive program of grade separations and grade-crossing protection. During this period, the casualty ratio [(injuries + fatalities) × 10^{18}]/(train kilometers × vehicle kilometers) dropped almost 70 percent, from 98.3 to 30.4. After 1930, there was a four-year period when the railroads stopped spending and almost nothing was done to improve grade crossings. Starting in 1935, some special federal programs to improve safety at railroad-highway grade crossings were initiated and carried forward to the war period. After the war, grade-crossing work was again resumed, and substantial amounts of money from federal-aid highway programs were used (9).

The Federal-Aid Highway Act of 1944 amended the law to provide special funding ratios for projects that would eliminate railroad-highway grade-crossing hazards on the federal-aid system. Under the provisions of this law, as much as 100 percent of the construction cost and 70 percent of the cost of right-of-way acquisition of such projects can be paid from federal funds. As much as 10 percent of the total federal-aid highway system funds apportioned to each state in any year can be spent by using the above ratios. Additional grade-crossing projects can be undertaken at the regular (usually 70 percent) funding ratio (9).

Increased congressional interest in railroad-highway safety and in the urban railroad problem led to the Railroad Safety Act of 1970 and the Highway Safety Act of 1970. These acts required the Secretary of Transportation to make a comprehensive nationwide study of railroad-highway grade-crossing safety (3) and report his recommendations to Congress (10).

The Highway Safety Act of 1973, as amended by the

Highway Safety Act of 1976, provides funding at a 90:10 ratio for grade-crossing safety improvements. Projects for crossings on the federal-aid system can be financed with Highway Trust Fund money, whereas off-system projects must be financed from general fund appropriations. At least half the funds authorized and expended must be for protective devices. In addition, the states are required to conduct and maintain a survey of all grade crossings that may require separation, relocation, or protective devices and implement a schedule for this purpose (9).

Problems with Current Programs

Possibly the greatest problem in improving safety at grade crossings has been in increasing the rate of installation of active warning devices. Although reliable figures on the rate of installation of these devices are difficult to obtain, it appears that the current rate is somewhat higher than that of the early 1970s. However, it continues to fall short of achieving the goal of 3000 active warning devices/year recommended by the 1972 report to Congress.

Active warning devices have traditionally been regarded as an effective means of significantly reducing the number of accidents at grade crossings. A study of the number of accidents at 1552 crossings in California before and after active warning devices were installed (11) supports this view. The study found that, in California, the relative accident expectancy at similar crossings that have standard devices is as follows: 1.00 for crossings with crossbucks, 0.33 for crossings with flashing lights, and 0.13 for crossings with automatic gates. For example, if 100 accidents were expected at a crossing where the warning device used was crossbucks, the installation of flashing lights would reduce the expected number of accidents to 33 and the addition of automatic gates would reduce it to 13. Other studies have shown that train-activated devices reduce accident severity in addition to reducing accidents.

Rates of accident severity by type of warning device [according to data given in Federal Highway Administration (FHWA) Notice N5120.3 of November 1975] are given below:

	Fatalities per Accident		Injuries per Accident	
Device	Rural	Urban	Rural	Urban
Crossbucks	0.32	0.13	0.82	0.55
Flashing lights	0.19	0.10	0.42	0.45
Automatic gates	0.09	0.04	0.27	0.28

Relocation and Consolidation Programs

A joint report written by FHWA and the Federal Railroad Administration (FRA) in 1976 (12) discussed the nature of conflicts between urban areas and railroads and estimated the magnitude of the problem in the United States. By using an analysis performed by the Stanford Research Institute, estimates of $1.8-1.9 billion were given for the cost of consolidation and relocation projects in which benefits could be expected to exceed costs.

As part of its study, FHWA conducted a survey of the states and railroads to determine the number of projects that had been completed since 1950 and the number that are currently in some stage of planning. The results of this survey are given in Table 1(8, p. 52).

Section 163 of the Federal-Aid Highway Act of 1973 authorized 12 cities to develop demonstration projects for relocation of railroad lines and/or elimination of railroad-highway grade crossings. The Federal-Aid Highway Amendments of 1974 and the National Mass Transportation Assistance Act of 1974 each added one additional city. Four more cities were added by the Federal-Aid Highway Act of 1976, and the 19th and final city was authorized by the U.S. Department of Transportation and Related Agencies Appropriation Act of 1977. As of December 1978, $548.4 million had been authorized for the demonstration cities.

Table 2 (13) gives the cities that are currently included in the demonstration project and their share of obligated federal funds as of December 31, 1978. The table also includes 1978 cost estimates for each of the projects.

HINDRANCES TO FUTURE PROGRESS

There are many obstacles to implementing solutions to conflicts between railroads and urban areas. These obstacles can be grouped into four categories: financial, institutional, statutory, and operational.

Financial Obstacles

High Cost of Projects

Perhaps the single most important obstacle to relieving conflicts between urban areas and railroads is the high cost associated with these types of projects. Estimates of implementation costs for individual projects in the FHWA demonstration program run as high as $114 million.

If relocation must be accomplished by establishing a new railroad corridor, the cost will be high, particularly if new right-of-way must be purchased. In addition to the high cost of land for right-of-way in urban areas, construction costs of approximately $465 000/track-km ($750 000/track mile) are not uncommon. If other structures and facilities are required, costs can be even higher. To get full benefit from the relocation, it is usually necessary to provide for grade separation of the highways that cross the railroad right-of-way. One grade separation can cost as much as several million dollars. Additional costs arise from the use of track and signal devices required to connect the new corridor to the existing rail lines.

When railroad relocation is carried out by consolidating several railroads into an existing railroad corridor, costs are usually lower. Costs, in this case, are determined in part by the amount of connecting track needed, the sophistication of the signalization required, and the capacity and level of classification of the upgraded track.

Financial Condition of Cities and Railroads

According to the report of the Stanford Research Institute on urban railroad relocation (8), not more than 10 percent of cities that experience serious conflicts with railroads would be willing (or able) to contribute more than 10-20 percent of project costs. Much of the U.S. railroad system is suffering from problems similar to those of the cities. Maintenance and operating costs are increasing faster than revenues. The past practice of many railroads of deferring regular maintenance has contributed substantially to the problem. Poor track conditions, undependable motive power, and car shortages have also led to a general decline in service. Many railroads have been

Table 1. Summary results of survey of completed and proposed relocation projects.

Type of Project	Completed Projects Number	Cost ($000s)[a] Average	Total	Proposed Projects Number	Cost ($000s)[a] Average	Total
Relocation	69	3385	233 565	32	6 912	221 184
Consolidation	27	927	25 029	35	2 804	98 140
Combination relocation and consolidation	32	5554	177 728	45	12 659	569 655
Elevation	22	9716	213 752	15	9 887	148 305
Depression	20	5117	102 340	7	10 081	70 567
Relocation of yards and terminals	50	5637	281 850	21	17 069	358 449
Unspecified	7	4518	31 626	30	10 196[b]	305 867
Total	227		1 065 098	185		1 772 167

[a]In 1973 dollars.
[b]Average cost for all planned projects was used.

Table 2. FHWA Railroad-Highway Demonstration Program projects.

Type of Project	City	Federal Funds Obligated[a] ($000s)	Estimated Project Cost[b] ($000s)[c]
Relocation	Elko, Nevada	8 851	26 000
	Lincoln, Nebraska	1 998	31 600
	Wheeling, West Virginia	96	24 800
	Carbondale, Illinois	2 263	63 200
	East St. Louis, Illinois	2 881	21 100
	Springfield, Illinois	3 579	114 300
	New Albany, Indiana	2 575	2 700
	Brownsville, Texas/ Matamoros, Mexico	1 210	24 400
	Lafayette, Indiana	360	61 800
	Hammond, Indiana	589	57 000
	Metairie, Louisiana	251	40 000
	Augusta, Georgia	306	97 000
	Pine Bluff, Arkansas	255	55 400
Grade separation	Blue Island, Illinois	230	5 900
	Dolton, Illinois	210	4 500
	Anoka, Minnesota	2 987	3 600
	Greenville, Texas	353	6 200
	Sherman, Texas[d]		
	Terre Haute, Indiana	285	6 000
Total		29 279	645 500

[a]As of Dec. 31, 1978.
[b]Federal share is now 95 percent for all cities in the program.
[c]In 1978 dollars.
[d]Withdrew from the demonstration program.

forced into bankruptcy, and others are only marginally solvent.

Even the financially strong railroads are unable to provide much support for relocation projects. The poor condition of the industry as a whole [an average 1.26 percent rate of return on investment in 1977 (1)] has generally made it difficult for railroads to raise cash in the equity markets. Even if the railroads are able to regain the confidence of the private investment community, it is not likely that they would be eager to finance railroad relocation projects. Dollar benefits to the railroads from urban railroad relocation are generally small and can even be negative. (Railroad benefits average about 20 percent of total benefits for the FHWA demonstration cities that have filed financial reports. However, railroad benefits are as low as 1 percent of total benefits for one city.) There are benefits to be derived from improving the public image of railroads, but these benefits contribute little to financial integrity. Furthermore, if new equity became available to the railroads, they would be interested in funding projects of their own that would produce much higher rates of return than relocation projects.

Limited State and Federal Funding

A 1975 survey of several states indicated an unwillingness on the part of the states to finance urban railroad relocation projects (8). Most states reported that they were not even able to handle their current highway needs with existing funding and that additional funding sources would be needed for railroad relocation. Since 1975, the increased maintenance requirements of the aging Interstate and federal-aid highway systems have worsened the financial condition of most state highway departments and left little, if any, money for rail-related projects.

The federal government is seen by most officials to be the only entity capable of funding urban railroad projects. Congress has authorized the current Railroad-Highway Demonstration Program, but there is some question as to whether there will be sufficient funds to complete the demonstration program.

Inflation

Since the inception of the Railroad-Highway Demonstration Program in 1973, the United States has been in a period of high inflation, particularly in the construction industry. The Federal-Aid Highway Act of 1973 authorized a total of $90 million for its 12 designated demonstration cities. As of December 1978, FHWA estimated the costs of these projects to be more than $300 million. The current estimated total cost of the entire 18-city demonstration project is $646 million (in 1978 dollars). At current rates of inflation, this price tag could increase by as much as $60 million each year the projects are delayed.

Institutional Obstacles

Multijurisdictional Problems

In most railroad projects of any magnitude, the operating and environmental impacts generally extend to more than one locality. In some cases, project limits extend beyond state boundaries and even national boundaries (the Brownsville, Texas, demonstration project, for example, is a joint U.S.-Mexico undertaking). Agreement on a final plan can be difficult when individual jurisdictions have different goals, priorities, and resources. Local jealousies, unequal distribution of project benefits and negative impacts, and conflicting objectives can hinder the acceptance of a unified plan of action.

Governmental Conflicts

Localities not included in the FHWA Railroad-Highway Demonstration Program must compete for other limited federal and state funds. One of the most likely sources of funds for railroad relocation is the federal funds available for eliminating hazards at railroad-highway grade crossings both on and off the federal-aid highway system. These funds are administered by FHWA and are given to the state highway and/or transportation departments for allocation to specific projects. The selection criteria used at the state level in disseminating these funds may make it very difficult for an urban railroad project to qualify for these funds on a priority basis.

Intraindustry Competition

Railroads are private enterprises that operate in a regulated, but nonetheless competitive, environment. Railroads compete with other railroads as well as with other modes of transportation. Competitiveness between railroads often surfaces in relocation projects. Some of these projects involve the consolidation of rail lines of more than one company into a single corridor, which may require joint use of a right-of-way or even joint use of track. When two railroads share the use of the same track, one usually assumes the responsibility for operational control and maintenance while the other pays trackage-right fees to cover its share of the costs. Satisfactory agreements between railroads are sometimes difficult to negotiate.

Railroad-Labor Relations

Railroad-labor conflicts usually emerge in relocation projects that involve the elimination of a railroad yard or restructuring of a terminal area. Union contracts generally have written into them a specific reporting location for the employees. If a railroad yard is relocated or its use changes, it will probably be necessary to negotiate new contracts with the unions involved, particularly if the reporting location is changed.

Local Conflicts

In most railroad relocation projects, some segment of the population will be adversely affected. It is virtually impossible to relocate a railroad in an urban area without some dislocation. Residents who are close to the project but not close enough to be displaced may also be adversely affected.

Negative impacts can evolve from the visual intrusions, vibration, noise, air pollution, and dangers associated with the higher volumes of highway traffic that result from altered travel patterns. Increased volumes of train traffic because of rail-line consolidation can have similar impacts on homes and businesses in the vicinity of the tracks. Bridges and other structures associated with relocation projects can affect the aesthetics of an area and, in doing so, lower property values. Strong neighborhood opposition to relocation plans may develop and result in the slowing down or termination of the project. In many cases, the philosophy is, "Let's get the railroad out of downtown, but don't put it in my neighborhood."

Lack of Lead Agency

Railroad relocation projects have no strongly unified proponents at the state or federal level. The existing congressionally sponsored demonstration program has been criticized for its special-interest orientation.

One of the main reasons for the lack of unified support for railroad relocation has to do with the nature of project benefits. Most railroad relocation projects are characterized by a wide dispersion of benefits and low benefit/cost ratios (0.65-1.55 for the FHWA demonstration cities). The benefits include savings in highway-user travel time, reduced potential of highway-railroad grade-crossing accidents, improved mobility for emergency vehicles and other highway users, reduction or elimination of the frustrations associated with waiting for trains to clear grade crossings, removal of rail facilities that are contributing to urban blight, removal of barriers between neighborhoods, improvement of area aesthetics, release of urban land for redevelopment, increased tax base, opportunity for economic development and urban renewal, improvement in railroad operations and in the public image of railroads, and a reduced probability of major catastrophies resulting from accidents involving hazardous materials. In most relocation projects, the benefits that accrue to any one segment of the population are not sufficient in themselves to justify funding the project. Although the total aggregated benefits may justify project costs, the disaggregated benefits are not great enough to get the attention of special-interest groups.

Statutory Obstacles

Statutory Limitations of Local Governments and Authorities

Local government units receive their legislative and operating authority from the state. Therefore, the statutory abilities of local governments to carry out railroad relocation projects vary from state to state. The problems that typically arise in connection with relocation projects are problems of debt limits and taxing abilities.

The amount of bonded indebtedness that a municipality can carry is usually limited to a set percentage of the municipality's tax base. For most communities, it is unrealistic to assume that relocation projects can be funded by floating municipal bonds. The report by Moon (8) estimates that the costs of railroad relocation projects, as a share of municipal outstanding debt, average from about 30 percent for communities of 50 000-100 000 to 141 percent for communities of 5000-10 000. Some areas have avoided the indebtedness limit by establishing a special-purpose authority to conduct the relocation project. Such authorities have their own indebtedness limits, which can be used entirely for their stated special purpose.

Interstate Commerce Commission and State Public Utility Commission Requirements

Because of their heavy involvement in interstate commerce, railroads are closely controlled by the Interstate Commerce Commission (ICC). Any modification of railroad operations in a community that is substantial enough to benefit that community will undoubtedly require ICC approval.

The ICC approval process can be lengthy, particularly if any opposition to the project exists. In railroad relocation projects, it is sometimes proposed that rail service to local industries located on light-density lines be eliminated. Because of the prospects of increasing freight costs, these industries may choose to plead their case before ICC, which can delay or even block the relocation project. The Railroad Revitalization and Regulatory Reform Act of 1976 attempted to speed up the ICC process by stipulating maximum time limits for the ICC approval process for railroad mergers, consolidations, and joint use of tracks or other facilities.

In addition to the ICC requirements, most relocation projects contain elements that come under the jurisdiction of state public utility commissions (PUCs) or similar organizations. Typically, highway-railroad grade crossings are controlled by the state PUC, as are railroad abandonments and consolidations.

Railroad Title Problems

In some relocation projects, railroads have had difficulty in producing clear titles for the lands that they want to sell. Sometimes the railroad holds only an easement for the property that is valid only if the property is used for railroad purposes. Sometimes the railroad's ownership of the land is subject to reversion to the previous owner's heirs if the railroad is abandoned. The problems of reversionary interest, easements, and other legal clauses in the original transfer documents add considerably to the cost and time involved in acquiring railroad rights-of-way. In some states, condemnation may be necessary in order to obtain clear title.

Another problem occurs when a railroad is not able to define clearly what land it actually owns. Conflicting deed descriptions, nonexistent deeds, and undocumented right-of-way maps add to the problem of obtaining clear title to railroad lands. Currently, much of the railroad property in the Northeast and the Midwest is, in some way, under the control of bankruptcy-court-appointed trustees. Sale of these properties must be approved by the trustees, which may be difficult. Even railroads not involved with bankruptcy procedures may have trouble disposing of land because of mortgage restrictions. In certain loans made to railroads, land has been used as collateral, and it cannot be sold without restructuring the loan agreements, which generally results in higher interest being charged to the railroads.

Operational Obstacles

A feasible plan for railroad relocation must consider the needs of both the urban area and the railroads. The general public does not adequately understand railroad operations. Professionally trained urban planners usually have little experience in railroad operations and have historically tended to plan around the railroads. On the other hand, consultants sometimes hire former employees of railroad engineering departments, whose perspective is often very narrow.

The tendency of people who are not experienced in railroad operations is to oversimplify operational needs. Problems arise when railroad officials reject community-generated relocation plans because of technical deficiencies. Community planners and engineers are unable to separate justifiable railroad operational needs from dispensable railroad demands. Thus, good, workable solutions to urban railroad problems are often difficult to achieve.

Train Speed and Length

A common complaint of communities is that trains take too long to clear a grade crossing and therefore cause massive traffic backups. In some cases, cities have passed laws that limit the time a train can block a railroad-highway intersection. Since most of these cities have also legislated maximum speeds for trains, their time restriction in fact becomes a restriction on train length, and this affects the railroad's ability to operate more efficiently by running long trains with small crews.

Joint Use of Track

Many railroad relocation plans call for two or more railroads to share the use of the same track or set of tracks. In addition to the problems that arise from railroad competition, other operating factors must be considered in planning for the joint use of track. Rail lines in a particular city represent only a small portion of a large, regional, integrated network of railroad operations. Trains are usually run on preset schedules to expedite the transfer or switching of cars from one train to another at transfer points. Changing schedules generally have systemwide effects on all railroads.

Conflicts may occur on consolidated trackage when line-haul operations are mixed with local deliveries and pickups. Stopping trains to set out or pick up locally generated traffic can cause delays to through trains. Track capacity is substantially reduced when local switching operations occupy the tracks for extended periods of time.

CONCLUSIONS

Conflicts between urban areas and railroads are widespread among cities nationally, although the nature and magnitude of the problems vary widely from city to city. Few systematic analyses of the benefits of railroad relocation have been conducted, but the few data that are available indicate that benefits are quite diverse and are usually not much greater than project costs. In some proposed projects, benefit/cost ratios have been less than 1.0. Consequently, urban railroad relocation projects have generally been given lower priority than other urban projects.

It is likely that conflicts between urban areas and railroads will intensify in the future and that this will cause an increased interest among cities in resolving these problems. The U.S. Conference of Mayors has recently become active in promoting rail relocation projects primarily aimed at economic development. To aid cities in their search for solutions to conflicts between urban areas and railroads, the conference has published a document that explains "the real story on rail relocation" (14).

New federal funding directed specifically toward urban-railroad relocation is very unlikely. The best way for cities to get federal assistance in solving railroad problems is to include railroad relocation as an element in other urban projects. For example, it could be an important part of a downtown urban renewal project or part of a highway-transit improvement project. Thus, railroad relocation could become a means by which to solve specific urban problems rather than a panacea for urban decay.

ACKNOWLEDGMENT

Some of the research for this paper was performed while I was employed by the Federal Railroad Administration in its summer professor's program.

REFERENCES

1. Yearbook of Railroad Facts. Assn. of American Railroads, Washington, DC, 1978.
2. Summary Statistics of the National Railroad-Highway Crossing Inventory for Public At-Grade Crossings. Federal Railroad Administration, U.S. Department of Transportation, 1977.
3. Report to Congress: Railroad-Highway Safety—Part 1: A Comprehensive State of the Problem. Federal Railroad Administration and Federal Highway Administration, U.S. Department of Transportation, Nov. 1971.
4. Rail-Highway Grade-Crossing Accident/Incident Bulletin. Office of Safety, Federal Railroad Administration, U.S. Department of Transportation, No. 43, 1977.
5. 1975 Societal Costs of Motor Vehicle Accidents. National Highway Traffic Safety Administration, U.S. Department of Transportation, Dec. 1976.
6. C. V. Lyon. Myths and Facts About Railroads in America. AREA Bull., Vol. 79, No. 668, July 1978, pp. 311-314.
7. A Prospectus for Change in the Freight Railroad Industry. U.S. Department of Transportation, Oct. 1978.
8. A. F. Moon. Urban Railroad Relocation: Nature and Magnitude of the Problem. Stanford Research Institute, Menlo Park, CA, Rept. RP-31, Vol. 4, April 1975.
9. Grade-Crossing Task Force Report. Federal Railroad Administration and Federal Highway Administration, U.S. Department of Transportation, Nov. 21, 1977.
10. Report to Congress: Railroad-Highway Safety—Part 2: Recommendations for Resolving the Problem. Federal Railroad Administration and Federal Highway Administration, U.S. Department of Transportation, Aug. 1972.
11. The Effectiveness of Automatic Protection in Reducing Accident Frequency and Severity at Public Grade Crossings in California. California Public Utilities Commission, Sacramento, 1975.
12. Railroad Consolidation and Relocations in Urban Areas: Report of the Secretary of Transportation to the Congress of the United States. Federal Railroad Administration and Federal Highway Administration, U.S. Department of Transportation, March 1976.
13. Fifth Annual Report on Railroad-Highway Demonstration Projects. Federal Highway Administration, U.S. Department of Transportation, Dec. 1978.
14. The Real Story on Rail Relocation. U.S. Conference of Mayors, Washington, DC, May 1979.

Publication of this paper sponsored by Task Force on Railroad Operations.

Addendum

Robert Schumacher

Editorial comment: The first five papers in this Record were presented in Session 3 of the 59th Annual Meeting of the Transportation Research Board. The following remarks by a participant and the presiding officer were made in the course of that session.

Instead of spending so much money running an expensive train around a test track, such as the one at the Facility for Accelerated Service Testing (FAST), why couldn't those same tests be made along some selected main-line track under normal traffic? Mention has been made that wear accumulates 5-10 times faster at FAST. First, I know of no exceptional urgency for the results, and, second, is the wear rate that much faster than it is on a heavy-traffic freight line in the Northeast, on the Santa Fe or the Union Pacific?

Controlled conditions were also mentioned. What does this mean? Of what value are controlled conditions except as they replicate the actual conditions under which North American railroads operate?

The need to make measurements does not seem to be a justification, since I know of no measurements that could not easily be made between regularly scheduled trains.

Discussion

William J. Harris, Jr.

In response to Schumacher's question about testing on main-line track instead of a test track, the more rapid accumulation of data is important. For example, rail must be tested under 181 million-272 million gross Mg (200 million-300 million gross tons) of traffic before significant trends in behavior can be established. In revenue service, this may take 5-10 years before a new set of choices is possible. During that time, the character of the traffic may change, thus altering the experimental conditions. FAST is intended to introduce 3.7 million-4.5 million gross Mg (4 million-5 million gross tons) of traffic per week. Thus, significant data are available in 1 year rather than in 5 or 10, and the test track provides much earlier information on track component behavior than can be accumulated in revenue service. If improvements and problems can be clearly shown in 1 year rather than 10, great advantages can accrue to the railroad industry. There is a need for much more rapid progress in the evaluation of new technology so that it can be applied to improve transportation effectiveness and safety.

The second question raised by Schumacher refers to controlled conditions. There are many circumstances in which one new component, such as an advanced rail material, is used by one railroad and another new component is used in revenue service by another railroad. The differences in traffic are such that comparisons within several percent are not feasible. This is another of the basic reasons why FAST was established. The railroad industry representatives responsible for the planning of FAST determined that it was essential to have uniform traffic on the test track to permit more precise comparisons of behavior. The cars and the locomotives used at FAST are identical to those in use on U.S. railroads.

The nature of the failures in car and track components is similar to that encountered in service. We know that not all conditions of service are duplicated at FAST. For example, the speed is below track hunting speed, and any effects associated with hunting are not observed at FAST. Nevertheless, there are enough similarities between revenue service and FAST operations that useful results can be obtained.

In regard to Schumacher's third question, there is no possibility that the kinds of measurements required at FAST can be made in revenue service. This would mean that the train must be stopped in daylight for about 8 h to take data on rail, ties, tie fasteners, ballast, and other components. At FAST, cars exposed to uniform operating conditions are taken out of service at frequent intervals so that measurements can be made on wheels, truck components, and other systems in the cars. There is no way in which a track can be vacated long enough to permit these kinds of measurements or that cars can be exposed to uniform operations and measured in revenue service as they are at FAST.

Railroad interest in and support of FAST are demonstrated by the willingness of individual companies to make available locomotives, cars, track components, and personnel on a continuing basis.