TRANSPORTATION RESEARCH

RECORD

No. 1289

Rail

Lateral Track Stability 1991

Proceedings of a Conference
May 15–17, 1990
St. Louis, Missouri

A peer-reviewed publication of the Transportation Research Board

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1991

Foreword

There is general agreement within the railroad industry that two of the most significant changes to occur in track maintenance since the end of World War II witnessed the mechanization of many heretofore labor-intensive operations and the gradual introduction of continuous welded rail (CWR) in the place of jointed rail. In the first case, the transition was an orderly process with evolutionary changes building on earlier modifications. Current work equipment bears scant resemblance to its ancestors of forty years ago—surfacing machinery is a case in point. It would be an exaggeration, however, to suggest that the integration of CWR into the many thousands of miles of track in which it performs today was well ordered. On the contrary, many failures occurred in the thirty- to forty-year span that started with the first tentative experiments to install genuine long lengths of CWR in track. It is now commonplace for CWR track to do its job, day in and day out, without the lateral stability problems that formerly plagued track maintenance engineers. This level of understanding, admittedly not perfect, is the result of a determined effort on the parts of researchers from government, industry, and academia and of practical track maintenance people to comprehend the mechanics of laterally unstable CWR track and to develop guidelines to be used by track workers to avoid lateral instability. The process of study, refinement, and practical application has been going on for almost twenty years in the United States, longer in some other countries, with the result that the failure rate of CWR track has reached an all-time low. Failure in this context is typified by the sudden lateral shift of track to relieve compressive force buildup in CWR, commonly called buckled track. To the members of the TRB Committee on Railway Maintenance it appeared that a useful service could be performed by providing a forum in which the state of the art in lateral track stability analyses could be described, along with measures adopted by track maintenance engineers to translate theory into practice. Such a conference was, in fact, held. It stimulated a rewarding level of interest, and the presentations of guest speakers at this event form the contents of this Transportation Research Record.

It appeared desirable in the design of the format of the conference to gather some insight into the ways in which the mechanics of lateral track instability were investigated abroad to complement discussions of the same topic by domestic investigators. The papers of Cervi, Miura, and Hagaman outline the approaches taken by the French National Railroads, the Railway Technical Research Institute in Japan, and the Queensland Railways in Australia, respectively.

Kish and Samavedam, examining different aspects of lateral track stability, take up track stability theory as it is widely understood in the United States and offer the results of tests that tend to confirm the basic assumptions.

Thompson offers a paper in which the efforts of one railroad to translate theory into practical advice and instructions for the work force are presented.

Procedural errors in the installation and maintenance of CWR are examined in detail by Ferguson, who relies on an analysis of train derailments caused by buckled track (CWR failures) to point out what went wrong over a period of years and how these errors can be circumvented.

The authors of the final four papers—Webb, Willbrant, Wickersham, and Ogden—are representatives of railroads that have been determined, from reports submitted over the last five years to FRA, to have exemplary records in preventing derailments caused by buckled track. Presented in some detail are statements of the methods and procedures advocated by railroads that have a history of success in avoiding stability problems with CWR track.

This Record is believed to be unique in the way in which it makes available, in one source, a synthesis of theory and practice, the understanding and use of which can enable confident and successful maintenance of CWR track.

William B. O'Sullivan
Federal Railroad Administration
Secretary, TRB Committee on Railway Maintenance

Transportation Research Record 1289
Price: $18.00

Subscriber Category
VIII rail

Printed in the United States of America

Library of Congress Cataloging-in-Publication Data
National Research Council. Transportation Research Board.

Lateral track stability 1990 : proceedings of a conference May 15–17, 1990, St. Louis, Missouri.
p. cm.—(Transportation research record ISSN 0361-1981; 1289)
"A peer-reviewed publication of the Transportation Research Board."
ISBN 0-309-05068-5
1. Railroads—Track—Congresses. I. National Research Council (U.S.). Transportation Research Board. II. Series.
TE7.H5 no. 1289
[TF240]
388 s—dc20
[625.1'4] 91-6273
CIP

Sponsorship of Transportation Record 1289

GROUP 3—OPERATION, SAFETY, AND MAINTENANCE OF TRANSPORTATION FACILITIES
Chairman: H. Douglas Robertson, University of North Carolina

Maintenance Section
Chairman: Jimmy D. Lee, North Carolina DOT

Committee on Railway Maintenance
Chairman: Mike Rougas, Association of American Railroads
Secretary: William B. O'Sullivan, Federal Railroad Administration
David M. Coleman, Ronald H. Dunn, Amir N. Hanna, Anne B. Hazell, Thomas B. Hutcheson, Robert A. Kendall, Richard G. McGinnis, Willis H. Melgren, Daniel B. Mesnick, Guenther W. Oberlechner, Albert J. Reinschmidt, Ernest T. Selig, William J. Semioli, Harry E. Stewart, Vincent R. Terrill, William C. Thompson, Donald R. Uzarski

Elaine King, Transportation Research Board staff

The organizational units, officers, and members are as of December 31, 1989.

Railways of Australia Track Buckling Project

B. R. HAGAMAN

The Queensland Railways undertook from 1986 through 1988 a major civil research project on track buckling on behalf of the Railways of Australia. From this project was developed a system that enables a railway system to reduce the probability of track buckling on its line sections. The elements of the track buckling prevention system, neutral temperatures of rail, theoretical aspects of safe operating temperatures, and the use of the Association of American Railways theoretical track buckling model to investigate the stability of a track standard are discussed.

Buckling of tracks has been a particular concern to engineers and administrators at the Queensland Railways during the last decade, and early in 1986 it was recognized that positive action was necessary to reduce its occurrence.

In July 1986, the Queensland Railways were commissioned by the Railways of Australia to undertake a research project on track buckling. The basic objectives were to (a) review the information available from within the state railway systems on track buckling, with particular emphasis on causes and procedures currently used to reduce its probability of occurrence, and (b) formulate recommendations and develop a practical track buckling prevention system to reduce the probability of track buckling occurrences.

A Railway of Australia report provides specific details of the project, the findings, and the track buckling prevention system (*1*). The project, as discussed by Hagaman and Heywood (*2*) and Hagaman and Kathage (*3*), was undertaken in two stages, each involving development, trial, and refinement of the track buckling prevention system. Validation of the prevention system was undertaken using the findings of overseas research, parametric studies using an analytical track buckling model, track buckling statistics collected in Australia, and the trial of the system in Queensland and Western Australia.

RAILWAYS OF AUSTRALIA PROJECT

Stage 1 of the project involved

- A review of existing maintenance practices on tracks with continuous welded rail (CWR), long welded rail, and short jointed track, particularly maintenance and operational procedures adopted during critical high ambient temperature conditions;
- A review of existing track buckling statistics available from Australian states and collection of additional data for validation of the buckling prevention system;
- Examination of existing track stability rating systems;
- A literature review; and
- The development and trial of an empirical track buckling prevention system based on the derived track stability rating and recommended maintenance and operational practices.

Stage 2 involved

- Validation of the buckling prevention system from a practical view,
- A review of existing buckling models and theoretical work,
- Implementation of the refined system on selected track districts in all states during the 1987–1988 summer, and
- Examination of buckling-related derailments and traffic operating practices.

Extensive monitoring was conducted after the completion of the project, particularly in Queensland, to evaluate the worth of the study in reducing track buckling occurrences and the practicality of the track buckling prevention system.

TRACK BUCKLING PREVENTION SYSTEM

The track buckling prevention system is designed to reduce the likelihood of buckling by allowing maintenance staff to perform their duties according to the system's main components:

1. The Maintenance Timetable, which indicates the recommended track work to be performed each month;
2. The Management Guidelines, which are a set of instructions to crews describing the preparations, precautions, and follow-up actions to be taken when performing various types of track work;
3. The Track Condition Report, which is produced from an annual inspection and provides a list of locations requiring remedial work to avoid buckling, along with the relative probability of buckling at each location;
4. The Track Maintenance Progress Report, which is a method of ensuring that items 1, 2, and 3 are being performed at the correct time of the year; and
5. An assessment of the adequacy of any existing track design standard for buckling resistance.

The field trials have shown that the timing of the various steps of the track buckling prevention system is fundamental

Queensland Railways, Railway Center, 305 Edward St., Brisbane, Queensland 4000, Australia.

to the system's success, and hence the Track Maintenance Progress Report is necessary. It is imperative that each step start at the correct time so that the remedial track work is completed before the start of summer.

The Maintenance Timetable and Management Guidelines together with the Track Maintenance Progress Report are sufficient to minimize the buckling problem in a railway system. This assumes that sufficient manpower and funds are available during the year to perform the work necessary to maintain the track to the chosen standard. However, the trend in track maintenance in all Australian railway systems in recent years has been toward an overall reduction in staff through the introduction of specialized migratory gangs and the use of outside contractors.

In response to this general need to minimize track maintenance costs, the Track Condition Report was developed. The aim of this report is to achieve a safe, stable track at minimum cost. An annual list of priority work that is used to direct maintenance staff to only those areas in need of urgent attention is produced.

The report procedure enables the district civil engineer to become aware of potential problem areas and to determine how best to organize limited maintenance resources to perform urgent work before the start of summer. The rating method used in the Track Condition Report does not attempt to assign absolute values for the buckling stability of the track, but simply locates the least stable spots within some convenient length of track. The rating is essentially a comparative method. It assumes that the rail system's chosen track standard for both curved and tangent track is adequate to hold the track securely against buckling through the expected temperature range, provided that the actual neutral stress temperature of the rail is within defined limits of the design neutral temperature. The adequacy of the design track standard can be investigated by use of an analytical buckling model.

Maintenance Timetable

Recommended in the timetable for track maintenance is the programming of maintenance activities most appropriate for each season. For example, maintenance of rail joints should be conducted before summer begins. Maintenance activities that disturb the track, such as resurfacing and resleepering, are recommended for the cooler months if track standards are not adequate.

The recommended timetable for maintenance activities in tropical and subtropical regions is shown in the following table.

Activity	*Timetable*
CWR stress adjustment	March to October
Rail joint maintenance	March to May, August to October
Expansion adjustment and anchor application	August to November
Lifting and packing near fixed track structures	March to September, remainder of year in early morning only
Resleepering	March to August, September and October in early morning only
Ballast profile and formation widening	All year, especially November to February

Management Guidelines

The regular maintenance practices that are normally required of track staff are formalized in the Management Guidelines. Aspects of both jointed and welded track are covered, including

- General maintenance of track components;
- Rail joint maintenance and surveillance;
- Rail gap and steel regulation of welded rail;
- Formation and ballast profile maintenance;
- Mechanized resurfacing, pulling, or lifting of the track;
- Operations performed on a face, such as mechanized resleepering or track relaying; and
- Use of rail anchors or indirect fasteners.

If track-disturbing work is performed at high temperatures, trains are subject to a speed restriction in order to allow reconsolidation of the ballast. Prohibition of certain maintenance activities is governed by the track standard, the work site temperature, and predicted climatic conditions for the following days. As such, the distribution of temperature forecasts and the monitoring of work site rail temperatures are important elements of the summer monitoring of areas prone to buckling.

Track Condition Report

For instances in which resources are not available to undertake remedial work in accordance with the maintenance timetable and the management guidelines, a track condition reporting and rating procedure was developed.

The Track Condition Report is produced from an annual inspection of the track condition against the design standard. Data from this inspection are entered into the dBASE III program RATING, developed for use on an IBM personal computer or compatible. From this program are produced a relative track buckling rating and a list in priority order of locations where remedial work is required to reduce the probability of track buckling on the line section. A sample output from the rating program is shown in Figure 1.

The track condition rating assumes that the rail system's design standards for both curved and tangent track are adequate to resist buckling. This assumption can be verified by use of an analytical track buckling model, and the rating cutoff level for urgent remedial work can be reduced accordingly for cases in which any deficiency in the design standard is identified.

The rating ranges from 0 to 100 and is a relative comparison of the actual track condition and the design track standard. A rating of 100 represents a high resistance to buckling. The rating automatically takes into account different track standards for curved and tangent track by comparison with the design standard. For example, a rating of 60 on curved track would represent higher track stability than a rating of 60 for tangent track, but would represent the same probability of buckling.

The track condition rating program RATING has been validated by in-track trials, examination of statistical data on buckling occurrences, and use of the Association of American

PRIORITY TRACK MAINTENANCE LIST

LINE	LOCATION (KM)		BUCKLING POTENTIAL	RESISTANCE RATING	TYPE OF WORK REQUIRED
** ACTION REQUIREMENT LOW PRIORITY.					
ATEST	12.0 -	12.5	3	91.0	,,,,
** ACTION REQUIREMENT URGENT					
ATEST	11.0 -	11.5	74	65.0	FREE JOINTS,ADJUST RAIL & ANCHOR,,BALLAST,SLEEPERS/FASTENERS
ATEST	10.5 -	11.0	73	68.0	FREE JOINTS,ADJUST RAIL & ANCHOR,,BALLAST,
ATEST	10.0 -	10.5	72	71.0	,ADJUST RAIL & ANCHOR,,BALLAST,
ATEST	11.5 -	12.0	67	83.0	FREE JOINTS,ADJUST RAIL & ANCHOR,,,

FIGURE 1 Sample output of the RATING program.

Railways (AAR) buckling model, TRACK. AAR provided the model for testing and evaluation during this project.

RATING was specifically designed to allow rail systems to modify the relative rating components or formula should alternative input parameters or resistance relationships be desired. At present RATING takes account of the following parameters for which data are required to be collected in the field during the annual track inspection.

- Rail
  - –Size,
  - –Temperature (for correct joint gap calculation),
  - –Length,
  - –Actual joint gap, and
  - –Frozen joints;
- Ballast
  - –Shoulder,
  - –Deficiency at shoulder,
  - –Deficiency at crib,
  - –Depth, and
  - –Type;
- Alignment horizontal curvature;
- Sleepers
  - –Type,
  - –Size, and
  - –Plating;
- Fasteners
  - –Type, and
  - –Defective percentage;
- Creep
  - –Rail, and
  - –Track;
- Support (local sleeper support); and
- Formation deficiencies.

Much of the data required from the field inspection is common from year to year, unless track upgrading has been undertaken, and typically 10 km/day of track can be inspected by a track supervisor.

The Track Condition Report was specifically designed to require minimum input by field staff. It is apparent from field trials that the key to acceptance and successful implementation lies in simplicity and in collecting only the minimum amount of data necessary.

In a trial of the rating system in Western Australia in one district in the summer of 1987–1988, of the 18 locations where track buckling was predicted from the rating, it occurred at 15 locations before remedial work could be undertaken.

TRACK Buckling Model

The AAR finite element track buckling model was developed by researchers at Clemson University to perform a nonlinear lateral deformation analysis on a railway track (*4*).

The program is general and takes into account

- Arbitrary rail properties, with the condition that both rails have the same properties;
- Arbitrary initial geometric imperfections;
- Rail to sleeper fastener torsional resistance, either linear or nonlinear;
- Lateral and longitudinal ballast resistance, either linear or nonlinear; and
- Arbitrary sleeper properties.

The program is capable of calculating postbuckling track deformations caused by thermal and mechanical loads and is capable of modeling tangent and curved track, including varying curvature, as in the case of a transition curve.

The model can be used to assess the adequacy of any given design track standard for resistance to buckling. The model's output was verified for a number of track structures through a parametric analysis and detailed examination of buckling statistics in Australia. Subsequently, the model was used to verify the track condition rating for a number of selected cases. However, further refinement of the assigned track stability rating would be necessary for gages other than 1067 mm.

NEUTRAL STRESS-FREE TEMPERATURE

As part of the Railways of Australia project, existing theoretical and experimental research work undertaken throughout the world was reviewed. Of particular note was the work of Whittingham (*5*), in which rail temperatures were recorded at hourly intervals over a 15-month period in Brisbane, Australia. From these data and Australian meteorological information for extreme maximum and minimum air temperatures, isotherms for expected maximum and minimum rail temperatures throughout Australia were established. Figures 2 and 3 represent the findings, from which a region's average rail temperature can be derived.

These figures form the basis of the project's neutral temperature recommendations on a regional basis throughout Australia. Neither the frequency distribution of maximum and minimum rail temperatures nor the local conditions are shown in Figures 2 and 3, but these factors need to be taken into account in the establishment of any region's design neutral rail temperature. The weighting of the design neutral temperature to a level greater than the region's mean rail temperature is recommended to reduce the probability of track buckling.

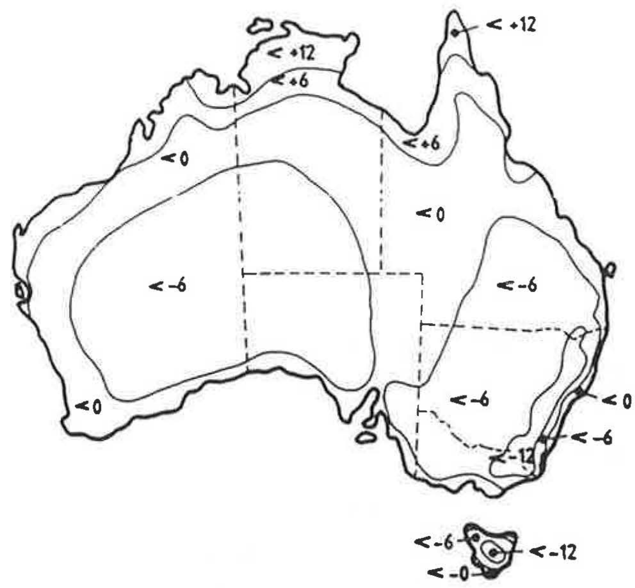


FIGURE 3 Minimum expected rail temperatures.

THEORETICAL ASPECTS

Safe Operating Temperatures

Central to the understanding of track buckling is the prediction of the critical buckling temperature for any particular design track structure or for a track structure with reduced buckling resistance resulting from normal in-service attrition.

Figure 4 shows the form of the typical temperature displacement curves for the buckling of a track structure and represents reduced track resistance for Curves 1 to 4. Parameters affecting the shape of the curves include curvature, alignment error, rail size, track stiffness, ballast resistance, and vehicle parameters. The dynamic margin of safety (DMS), as defined by Kish (*6*), represents the energy barrier on the temperature displacement curve that must be overcome before the track structure will buckle. The DMS against buckling equals the difference between the temperature increase above the neutral temperature to cause buckling (dT_B) and the safe temperature increase minima (dT_S). Figures 5 and 6 represent

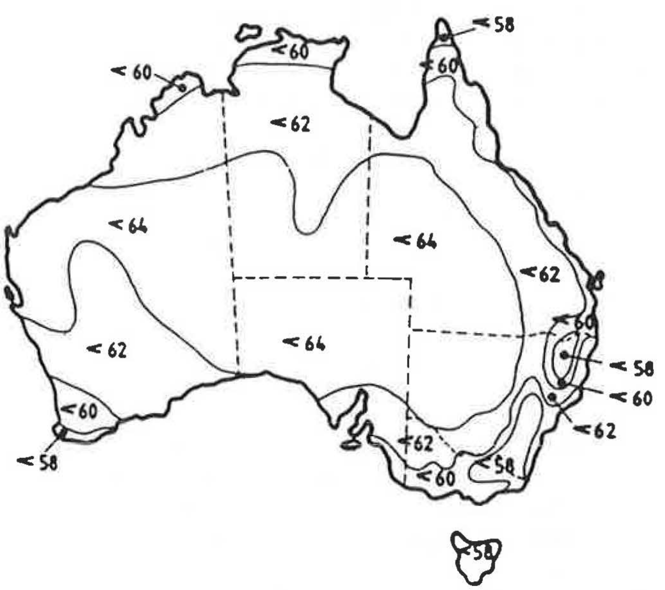


FIGURE 2 Maximum expected rail temperatures.

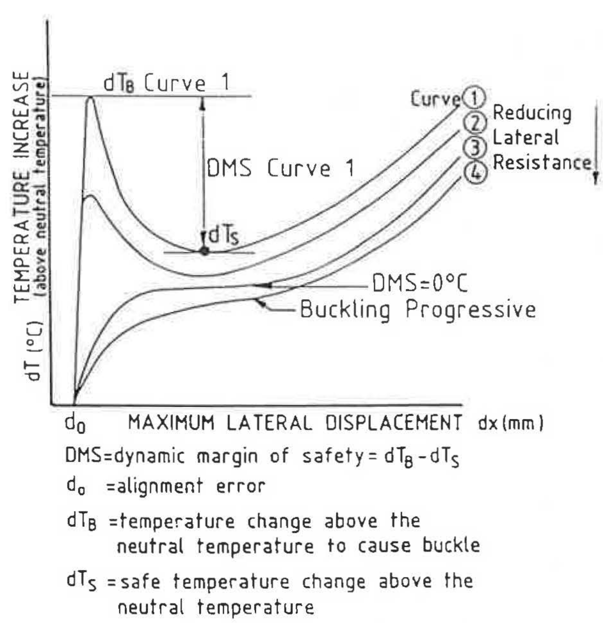


FIGURE 4 Typical temperature displacement curves.

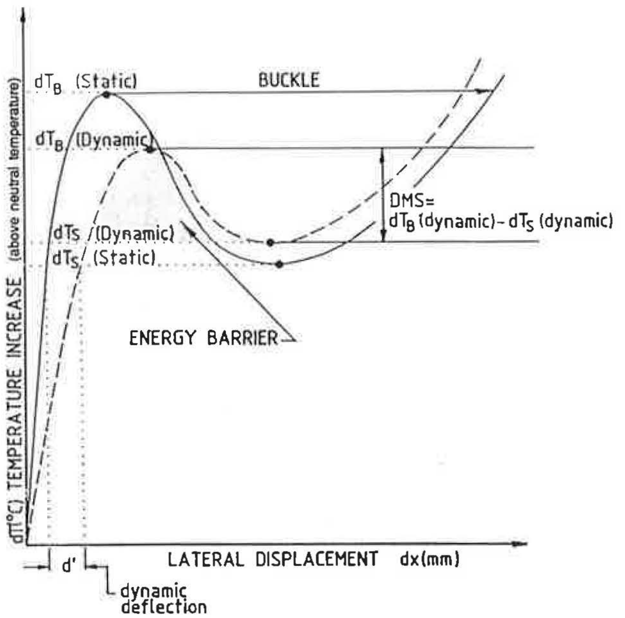


FIGURE 5 Buckling response for track with high buckling resistance.

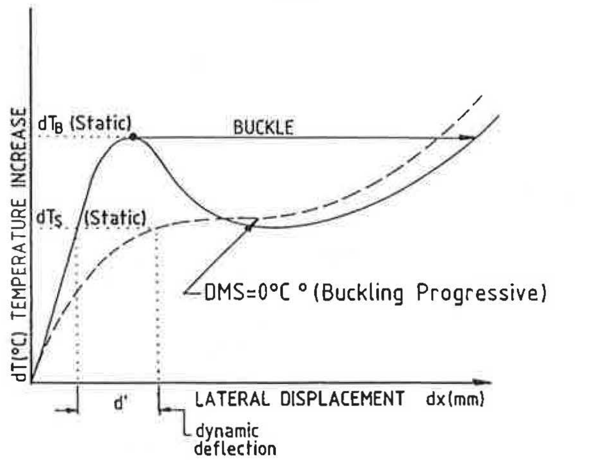


FIGURE 6 Buckling response for track with low buckling resistance.

the static and dynamic buckling responses for track structures with high and low resistance to buckling, respectively.

A track structure will buckle when the temperature exceeds the buckling temperature:

$$T_B = T_N + dT_B \tag{1}$$

where

T_B = buckling temperature,
T_N = actual neutral temperature, and
dT_B = the temperature increase above the actual neutral temperature to cause buckling.

The adequacy of any design track structure can be assessed by determining the dT_B and dT_S limits from an analytical model such as TRACK. The track's safe operating temperature (T_o) can be determined from these values. Comparison can then be made with a desired limit based on regional ambient conditions.

The following limits are recommended for the safe operating temperature T_o, based on the work of Kish (*6*) and the project findings:

When DMS $\geq$ 10°C,

$$T_o = T_N + dT_S \tag{2}$$

when DMS $\leq$ 10°C,

$$T_o = T_N + dT_B - 10°\text{C} \tag{3}$$

where

DMS = dynamic margin of safety = $dT_B - dT_S$,
T_o = safe operating temperature, and
dT_S = safe temperature increase above the actual neutral temperature.

These recommendations are represented in Figure 7. The 10°C margin that allows for dynamic loading and reduced lateral resistance following maintenance can be increased to take into account the reduced lateral stability of a track structure from the design standard due to attrition. The margin also allows for actual in-track neutral temperature variations from the design neutral rail temperature.

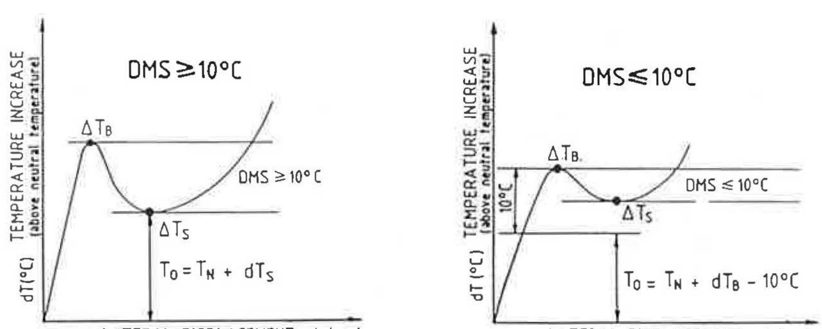


FIGURE 7 Recommended safe operating temperature limits.

Analytical Examination of Track Standards Using TRACK

The adequacy of any design track standard for stability in track buckling can be assessed using an analytical model such as TRACK. The process of assessment of existing track standards is a key element of the track buckling prevention system. The TRACK program was verified as suitable through examination of data on actual track buckling occurrences and by conducting a number of analyses of selected track standards using the program. In addition, parametric studies using TRACK were undertaken to examine a number of track standards.

It was found in the use of the analytical model of track stability that particular care was required in the analysis of the derived data and particular attention placed on selecting appropriate initial track misalignment values for dynamic deflection.

Summarized in Table 1 are the results of one of the parametric studies for the effect of rail size on a track standard's critical and safe buckling temperatures. These results are shown in Figures 8, 9, and 10 for concrete, steel, and timber sleepered track structures.

The track's safe operating temperature (T_o) has been calculated on the basis of a neutral temperature (T_N) of 35°C; however, alternative values can be substituted in Equations 2 and 3.

The effect of varying track gage for concrete sleepered track with a constant ballast profile and rail size is shown in Figure 11 for initial track misalignments of 10 and 45 mm.

TABLE 1 EFFECT OF RAIL SIZE ON CRITICAL AND SAFE BUCKLING TEMPERATURES

Concrete Sleepered Track (280 kg mass at 685 mm spacing)

Temperature (Change) °C		dT_B		dT_S		DMS		dT_O		T_O (dT_O + 35°C)	
Initial Misalignment mm		45	10	45	10	45	10	45	10	45	10
Rail Size kg/m	20	68.1	120	58.9	120	9.2	-	58.1	-	93.1	-
	31	55.1	120	52.9	120	2.2	-	45.1	-	80.1	-
	41	48.5	103.8	48.5	60.4	0	43.4	38.5	60.4	73.5	95.4
	47	45.9	96.7	45.9	58.4	0	38.3	35.9	58.4	70.9	93.4
	50	42.4	89.0	42.4	54.0	0	35.0	32.4	54.0	67.4	89.0
	53	44.5	93.9	44.5	59.9	0	34.0	34.5	59.9	69.5	94.9
	60	48.1	89.2	48.1	58.2	0	31.0	38.1	58.2	73.1	93.2

Steel Sleepered Track (7.5 mm thick section at 685 mm spacing)

Temperature (Change) °C		dT_B		dT_S		DMS		dT_O		T_O (dT_O + 35°C)	
Initial Misalignment mm		45	10	45	10	45	10	45	10	45	10
Rail Size kg/m	20	53.7	120	50.3	120	3.4	-	43.7	-	78.7	-
	31	44.8	98.0	44.8	57.3	0	40.7	34.8	57.3	69.8	92.3
	41	39.6	83.9	39.6	51.7	0	32.2	29.6	51.7	64.6	86.7
	47	37.9	78.8	37.9	51.4	0	27.4	27.9	51.4	62.9	86.4
	50	35.1	72.3	35.1	48.3	0	24.0	25.1	48.3	60.1	83.3
	53	40.8	76.9	40.8	50.9	0	26.0	30.8	50.9	65.8	85.9
	60	37.7	69.2	37.7	46.3	0	22.9	27.7	46.3	62.7	81.3

Timber Sleepered Track (115 x 230 x 2150 mm at 685 mm spacing)

Temperature (Change) °C		dT_B		dT_S		DMS		dT_O		T_O (dT_O + 35°C)	
Initial Misalignment mm		45	10	45	10	45	10	45	10	45	10
Rail Size kg/m	20	48.0	105.7	45.8	55.6	2.2	50.1	38.0	55.6	73.0	90.6
	31	40.8	87.3	40.8	51.8	0	35.5	30.8	51.8	65.8	86.8
	41	37.3	76.3	37.3	48.9	0	27.4	27.3	48.9	62.3	83.9
	47	36.3	71.8	36.3	47.9	0	23.9	26.3	47.9	61.3	82.9
	50	32.6	66.2	32.6	44.8	0	21.4	22.6	44.8	57.6	79.8
	53	37.1	70.4	37.1	46.7	0	23.7	27.1	46.7	62.1	81.7
	60	34.2	63.6	34.2	43.8	0	19.8	24.2	43.8	59.2	78.8

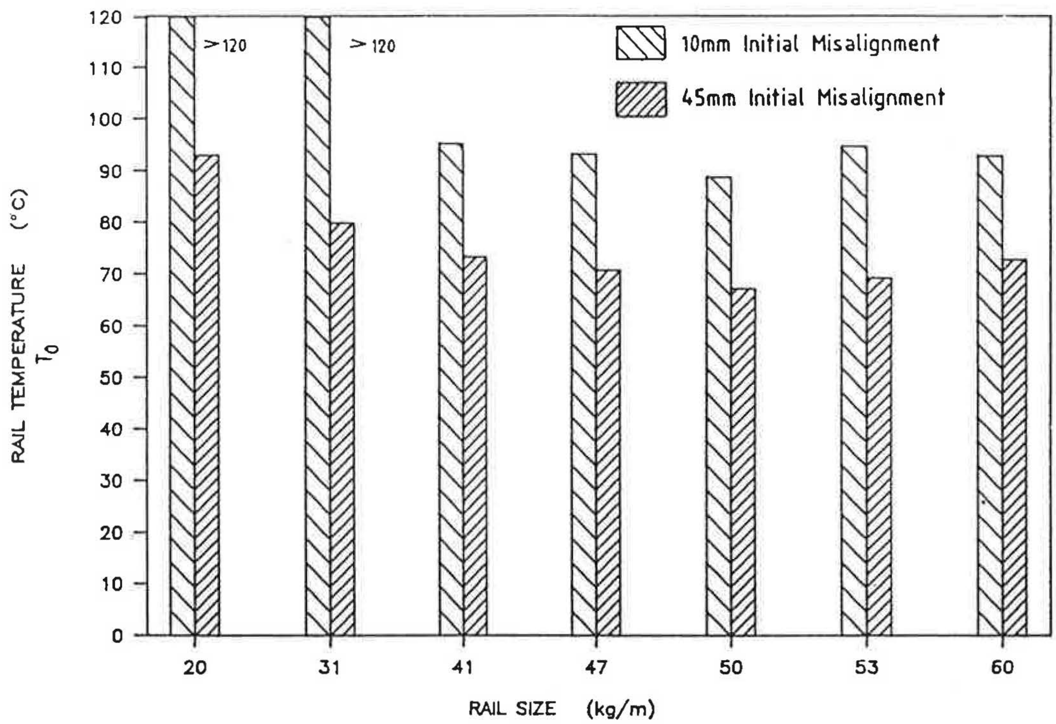


FIGURE 8 Effect of rail size on safe operating temperature—concrete sleepered track.

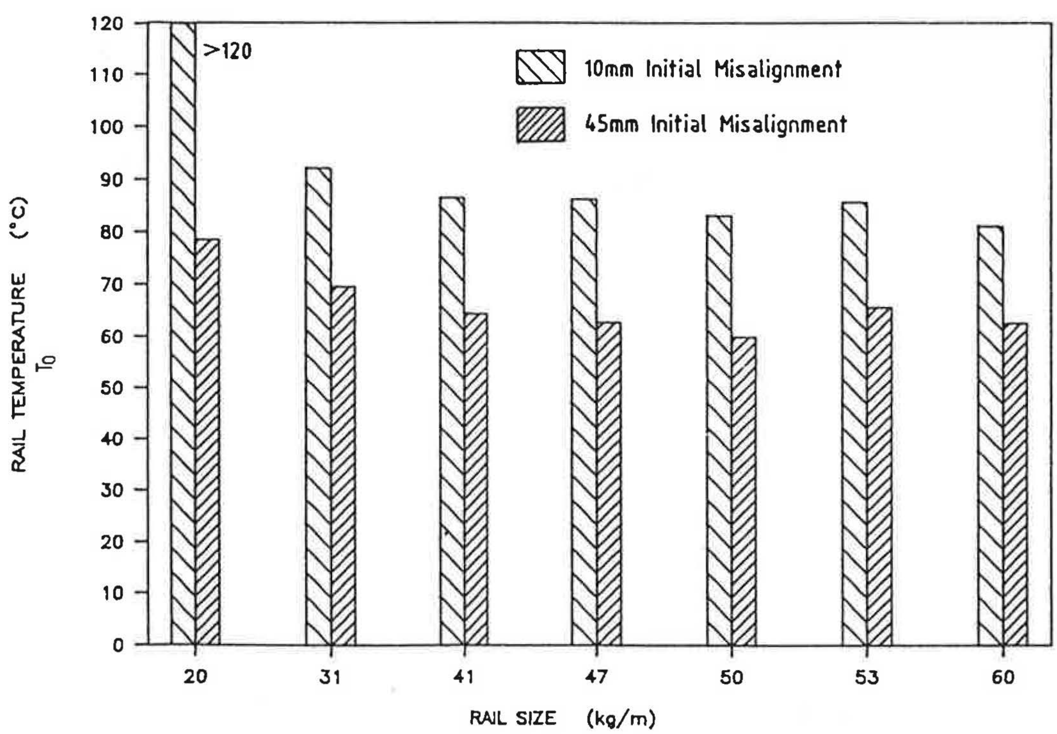


FIGURE 9 Effect of rail size on safe operating temperature—steel sleepered track.

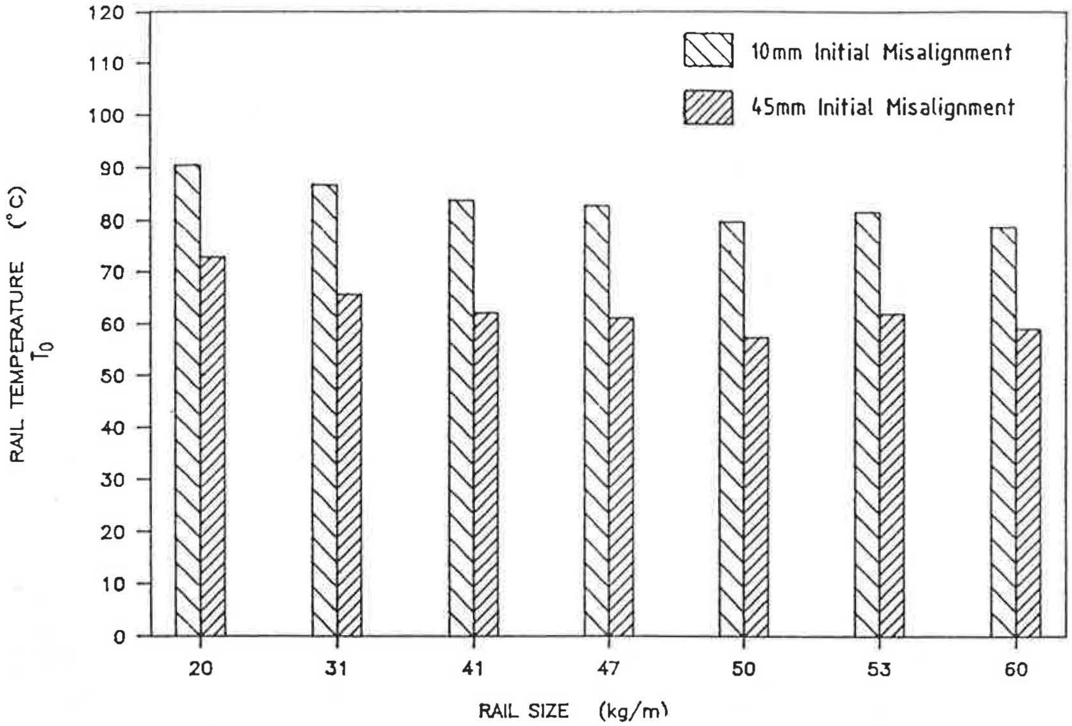


FIGURE 10 Effect of rail size on safe operating temperature—timber sleepered track.

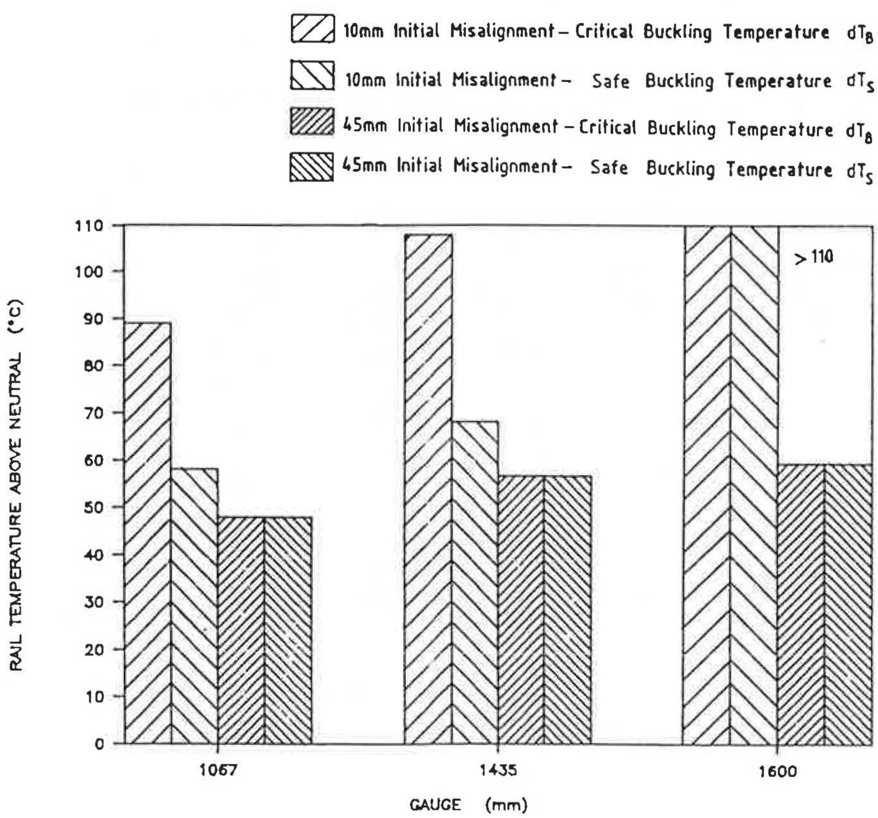


FIGURE 11 Effect of track gage on critical buckling temperature—concrete sleepered track.

POSTPROJECT ANALYSIS

Examination of the statistical data collected during the project and during the last 10 years in Queensland can give some insight into the effectiveness of increased emphasis on preemptive maintenance measures to combat buckling.

Figures 12 and 13 clearly show when the trackman must be most vigilant to detect occurrences. It is axiomatic that the greatest amount of buckling occurs in the summer, but the bias in the transition period from the cooler months and the mid-afternoon period confirms the practical experience of trackmen in Queensland.

Figure 14 shows the annual number of buckling occurrences in the Toowoomba District of southwest Queensland and demonstrates the effect of introducing mechanized maintenance procedures with a corresponding reduction in maintenance staff and the more frequent disturbance of the track and without the adoption of higher track standards or buckling prevention measures.

The measure of any project is whether it produces the required result. A clear reduction in the number of occurrences at the commencement of the project, even before significant feedback to field staff was affected, is demonstrated in Figure 15. It is clear that the increased emphasis that was placed on

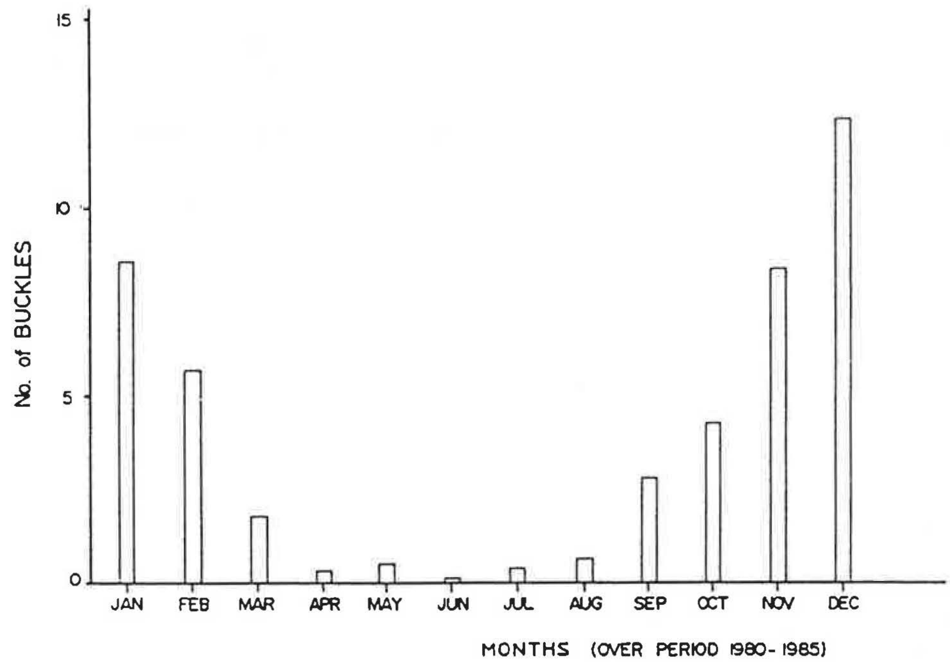


FIGURE 12 Distribution by month of track buckling in Queensland, 1980–1985.

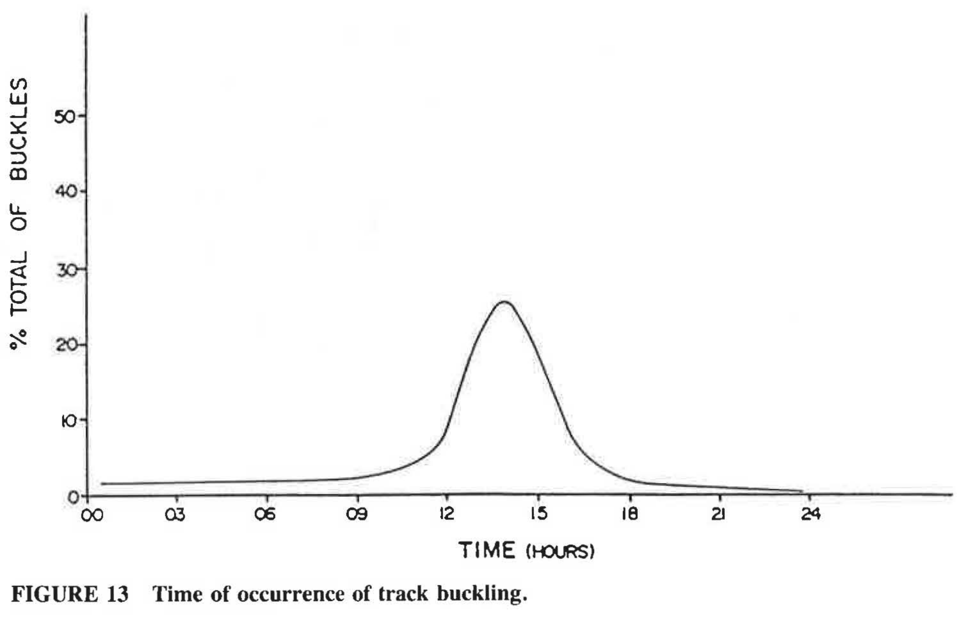


FIGURE 13 Time of occurrence of track buckling.

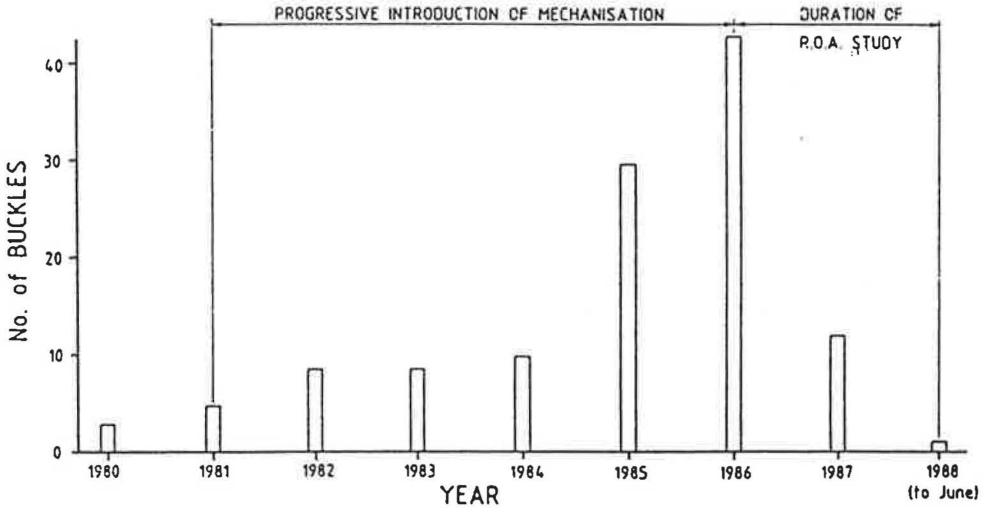


FIGURE 14 Annual distribution of track buckling in mechanized area in Toowoomba District.

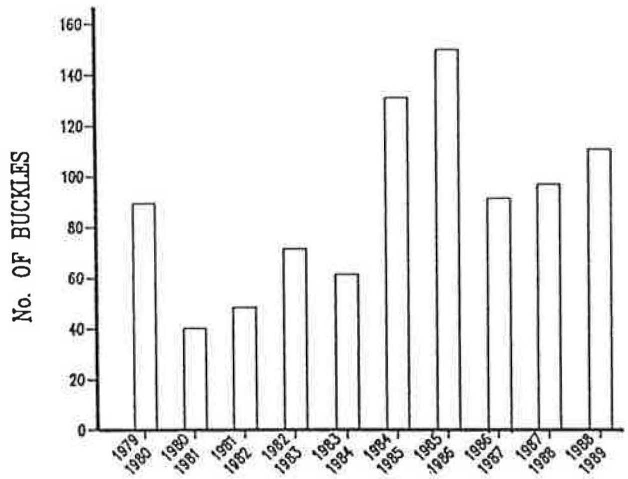


FIGURE 15 Annual track buckling occurrences in Queensland.

reducing buckling through the application of sound maintenance practices and the appropriate programming of work was a key factor.

CONCLUSION

An effective track buckling prevention system has been developed by the Queensland Railways on behalf of the Railways of Australia. The system enables a railway system to reduce the probability of track buckling on its line sections through an assessment of existing track standards, the adoption of maintenance guidelines, an annual track rating (if necessary), and the direction of maintenance resources to those areas identified as requiring urgent attention. Preemptive maintenance rather than proactive rectification was the underlying philosophy.

ACKNOWLEDGMENTS

The direction of the Railways of Australia Technology Development and Application Committee in the progression of this study and the permission of the Commission for Railways, Queensland Railways, to publish this paper are gratefully acknowledged.

REFERENCES

1. Track Buckling Project Technology Development and Application Committee. Report ROATDAC. Railways of Australia, Nov., 1988.
2. B. R. Hagaman and R. J. Heywood. *Queensland Railways Civil Research: Conference on Railway Engineering.* The Institution of Engineers, Australia, 1987.
3. B. R. Hagaman and L. O. Kathage. *Track Buckling: Seventh International Rail Track Conference.* Oct. 1988.
4. J. E. Jackson. *"TRACK" Users Manual—Version 2.0.* Association of American Railroads, Clemson University, Mechanical Engineering Department, Clemson, South Carolina, 1987.
5. H. E. Whittingham. *Temperature in Exposed Steel Rails.* Working Paper 40/1967. Commonwealth of Australia, Bureau of Meteorology, Department of Interior, June 1969.
6. A. Kish. *Track Buckling Workshop No. IV.* Federal Railroad Administration, Transportation Systems Center, U.S. Department of Transportation, 1987.

Thirty Years of Experience with Continuous Welded Rail on the French National Railroads

GÉRARD CERVI

For many decades, railroad technology was used to set up tracks with jointed rails and lengths in accordance with rolling technology and handling possibilities. With increasing loads and speeds and improvements in rolling, welding, and fastening technology, railroad engineers became interested in eliminating joints, which have drawbacks in the track and in controlling rising maintenance costs. A French railroad engineer in the early 1930s carried out his first studies and reflections. His conclusions and tests after World War II led step by step to 100 mi of welded track in France. The results of using welded rail are cost-effectiveness and greater comfort.

During the early development of railroad technology, rail lengths were limited by the rail manufacturing process, first to 1.5 m (5 ft), then to 8 m (26 ft), to 12 m (40 ft), to 18 m (60 ft), and finally to 36 m (118 ft). Today's rolling plants produce lengths of 36 m (120 ft) or even 72 m (240 ft).

After the end of World War II in 1945, new developments in rail welding on site and in plants and improvements in fastenings allowed continuous welded rail (CWR) technology to be developed and to become the worldwide standard for laying track. CWR is in constant development on the French railroad network, with an ever-increasing total mileage (Figure 1). These improvements are the results of a theoretical approach in 1932 and 1933, which was confirmed by theoretical and experimental research after World War II and developed as shown in Figure 1.

JOINTED TRACK BEHAVIOR

Rails are interrupted at regular intervals to allow expansion gaps to cope with changes in temperature. Variation in rail length is not due to a simple expansion of the material but to a mixing of expansion and stresses from the friction of the track on the ballast bed or the friction between the rail and the tie. Usually it is a composition of the two levels of friction according to the fastening quality and the ballast quantity. The compression force in a rail length is equal to

$$F = fo + \frac{L}{2} \times r$$

where

F = compression force,
fo = friction between rail and joint bars (joint assembly),
L = length of the rail, and
r = longitudinal friction coefficient of the track in the ballast.

Compression forces in a rail are shown in Figure 2.

Friction is not a linear function, and therefore length variations are phased out with variations of temperature. If the gap between two rails is suppressed, the rails become effectively continuous and the compression force increases according to the rising temperature.

$$F' = ES\,\alpha \cdot \Delta t$$

where

S = rail cross sectional arc
E = Young's modulus,
Δt = increasing temperature, and
α = steel expansion coefficient.

If this situation occurs, the compression force at the joint could be high, and the track stability has to compensate to avoid buckling. This track stability is mainly solicited in curves or misalignments where a transverse component of the compression could be a major factor. Moreover, in a jointed track the total inertia of the rail is cut and replaced by two joint bars, which are not a sufficient substitute, resulting in a weak joint. The track stability is always unfavorable in a jointed track, and it is important to keep sufficient gaps between rails to avoid stresses in the rails during hot weather, which involves inspection and maintenance.

From the vertical standpoint, a weakness in track stability can be more damaging to the surface of the track than a potential buckling for the same reason—lack of inertia.

Maintenance of maximum rail inertia is necessary to (*a*) reduce the maintenance costs; (*b*) improve passenger comfort; and (*c*) save the track components (rails, ballast, fastenings), maintaining a good leveling.

CWR DEFINITIONS

A CWR is an unlimited length of rail, interrupted only for technical, on-site reasons such as long span bridges. The rail

Société Nationale des Chemins de fer Français, Direction de l'Equipement, 17, rue d'Amsterdam, 75008 Paris, France.

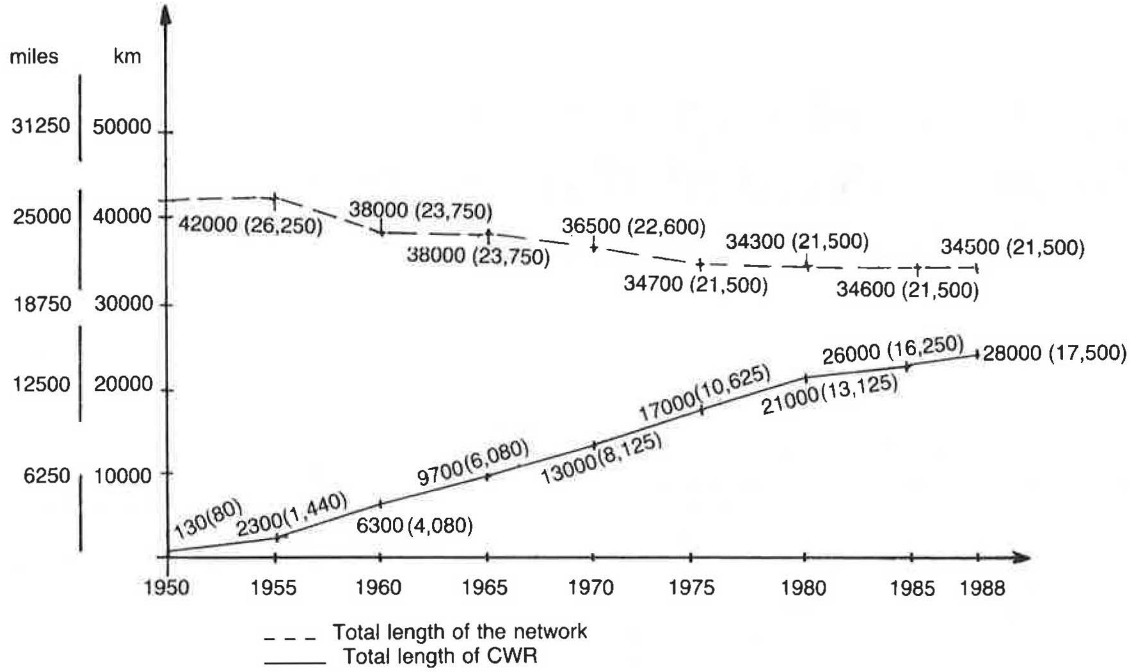


FIGURE 1 CWR on the French National Railroads.

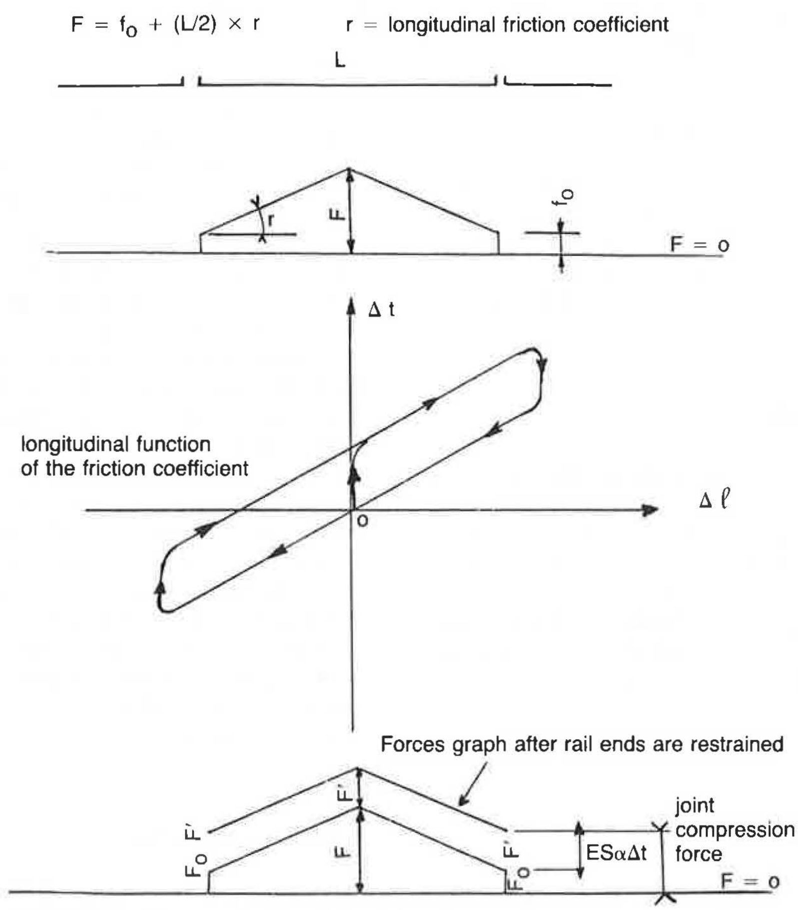


FIGURE 2 Compression forces in rail.

is secured on the ties by resilient fastenings, in order to keep the rail clamped to the tie under any circumstances. No movement between rail and tie is allowed; all friction and longitudinal displacements must be between the tie and the ballast. The longitudinal restraining force between the rail and tie must be more efficient than the longitudinal restraining force of the track in the ballast bed.

The track assembly (tie, rail, pads, fastenings) is laid on hard crushed stone—the ballast—with optimized quantities and specified profiles. In the major part of a length of CWR, variations of the temperature induce only stress variations (without movements), but in two areas of the CWR both stresses and length variations occur. These stresses (forces) and length variations must be contained by the track, and the quality of the track that copes with the necessity to avoid buckling is its stability, which primarily takes into account the transverse direction. The two areas of a CWR affected by longitudinal displacements according to variations in temperature are the ends.

The friction coefficient between the track (ties) and ballast restrains compression forces gradually, the total forces being blocked on lengths that are functions of the rail temperature changes (Δt) between the laying temperature or stress-free temperature (t_o) and the actual temperature of the rail (t_r).

CWR definitions are shown in Figure 3. In Figure 3, the compression force (F) in breathing zones 1 and 3 is

$$F = ES\,\alpha\Delta t - \int \frac{du}{dx} ES$$

The compression force in zone 2, the middle part blocked by the two breathing lengths, is

$$F = ES\,\alpha\Delta t$$

Under a simultaneous action of Δt and F, length/variation of the track element is

$$du = \left(\alpha\Delta t - \frac{F}{ES}\right) dx, \left(\frac{dl}{L} = \frac{\sigma}{E} = \frac{F}{SE}\right)$$

$$\frac{du}{dx} = \alpha\Delta t - \frac{F}{ES}$$

from which

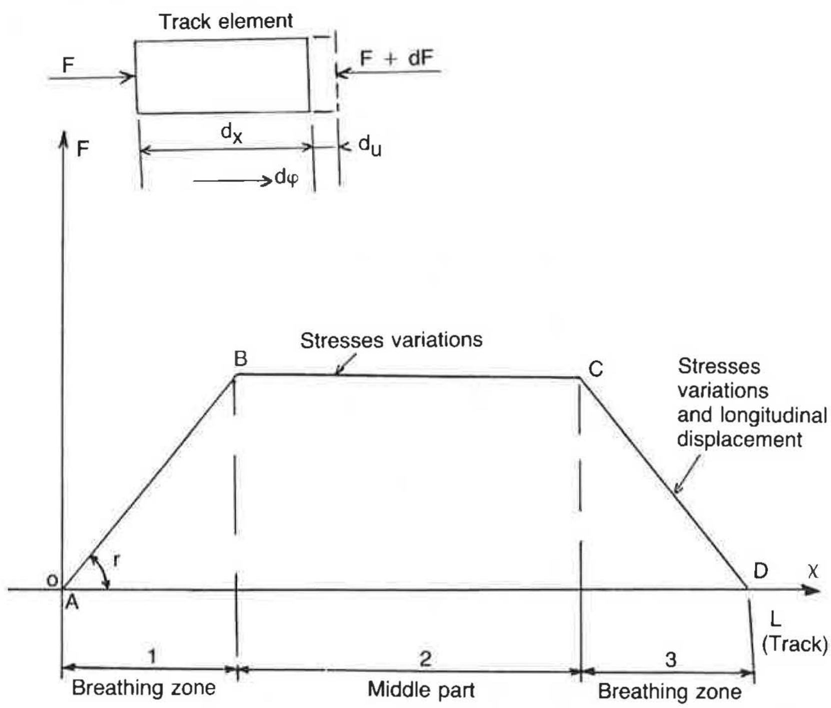
$$F = ES\,\alpha\Delta t - \int ES \frac{du}{dx}$$

The F graph is presented with a simplified function of the friction coefficient, which is normally a function of the displacement

$$\frac{d\phi}{dx} = f(u)$$

where

ϕ = friction coefficient,
u = track base displacements, and
x = length of rail.



$$1\text{-}3:\ F = ES\,\alpha\Delta t - \int \frac{du}{dx} ES \quad \text{or} \quad F = ES\alpha\Delta t - ES\int \frac{du}{dx}$$

$$2:\ F = ES\,\alpha\Delta t$$

FIGURE 3 CWR definitions.

The friction coefficient is shown in Figure 4. The simplified function $d\phi/dx = f(u) = \pm r$ is set up to simplify assumptions in the theoretical approach. A normal function is more complicated, but this approach is sufficient for a correct evaluation of the longitudinal phenomena.

If we imagine a rail that is laid stress free with sufficient accuracy, the longitudinal force distribution is presented in Figure 3. Both ends of the CWR are in movement, and points A and D are free of stress. Breathing lengths $A-B$ and $C-D$ experience stress (force) variations and movement. Between points B and C, only stress (force) variations occur.

The behavior of the CWR according to temperature variations is shown in Figure 5. Increasing temperature from the stress-free temperature to a higher temperature t_1 (t_o is the stress-free temperature),

$$F_1 = ES\,\alpha\,(t_1 - t_o),$$

with a linear connection to $F_1 = 0$ and with the simplified friction coefficient between ballast and ties $tg\,\alpha = r$.

If temperature t_2 occurs, the horizontal part of the F line will decrease to

$$F_2 = ES \cdot \alpha\,(t_2 - t_o)$$

or

$$F_1 - F_2 = ES\,\alpha\,(t_1 - t_2)$$

or

$$\Delta F = F_1 - F_2 = ES\,\alpha\,(t_1 - t_2)$$

In order to understand what happens in the breathing length it is assumed that the end of the CWR is restrained (i.e., no movement is possible). The graph will be parallel to the first one (t_1), with a gap.

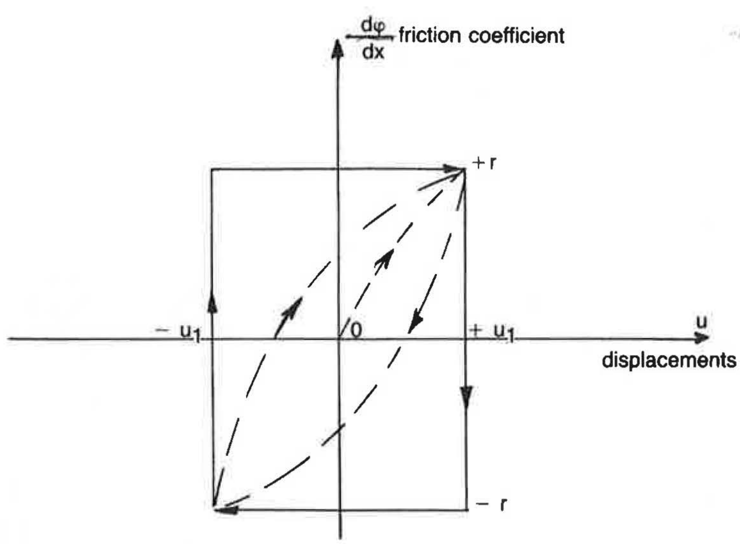
$$\Delta F = ES\,\alpha\,(t_1 - t_2)$$

If the restrained end is again considered free (i.e., there are no stresses at the end), all the points of the CWR between A and C will have tensile stresses, and the end will have movement between A and D toward the middle part. The graph shows irregularities in the breathing length after only one cycle in temperature. A total scope of the expansion zone behavior would show complete movement of the expansion zone end between two points according to the maximum temperature changes of the year. Between these two extreme positions only slight variations closely linked to the general sketch occur.

CWR BEHAVIOR AND TRACK STABILITY

In the middle part of the CWR, variations of temperature involve only stress variations, which give the rail a general state of balance comparable to a long beam under compression stresses in potential buckling situations.

A track is different, however, because of rail fastening assemblies and specified ballast layer profiles. This special rail-fastener-crosstie frame must withstand temperature stresses without noticeable geometric defects, which could affect traffic. The stability of the track allows it to keep its geometry during all the temperature variations throughout the year. Different factors confer stability to the track, including



$$\frac{d\varphi}{dx} = -\frac{dF}{dx}$$ is the real friction coefficient in relation with the displacement u

$$\frac{d\varphi}{dx} = \Delta f(u)$$

To simplify assumptions we use this simplified function $f(u) = \pm r$

FIGURE 4 Friction coefficient.

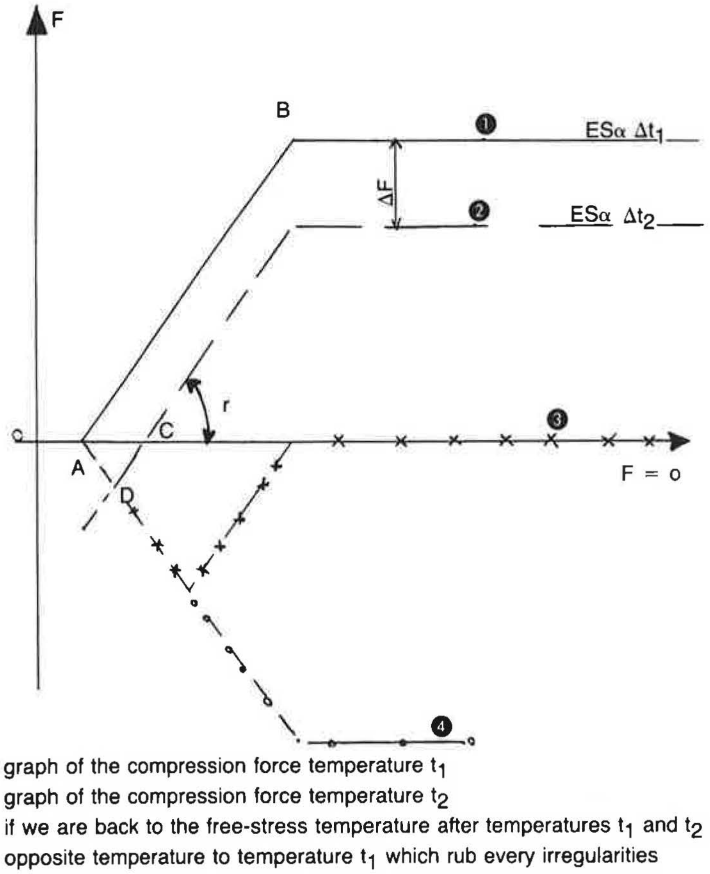


FIGURE 5 CWR behavior as affected by temperature variations.

- Rail-fastening assemblies;
- Ballast profiles, with designed depth, superelevated (heaped) shoulders, and ballast particle size (gradation);
- Track weight and rail inertia; and
- Track geometry quality (alignment imperfections).

An efficient rail-fastening assembly is necessary to keep the rail securely fastened on the tie to avoid any longitudinal movement between the rail and tie and to restrain the rotation of the rail base on the tie (torsional resistance). This value is defined by a torque per meter of track (statistical mean of tests), in accordance with the angle of rotation. Movement of CWR ends is shown in Figure 6. The limits of the expansion zone are demarked by $+ u_1$ and $- u_1$. C, the torque, is

$$C_{\text{torque}} = K\alpha$$

where

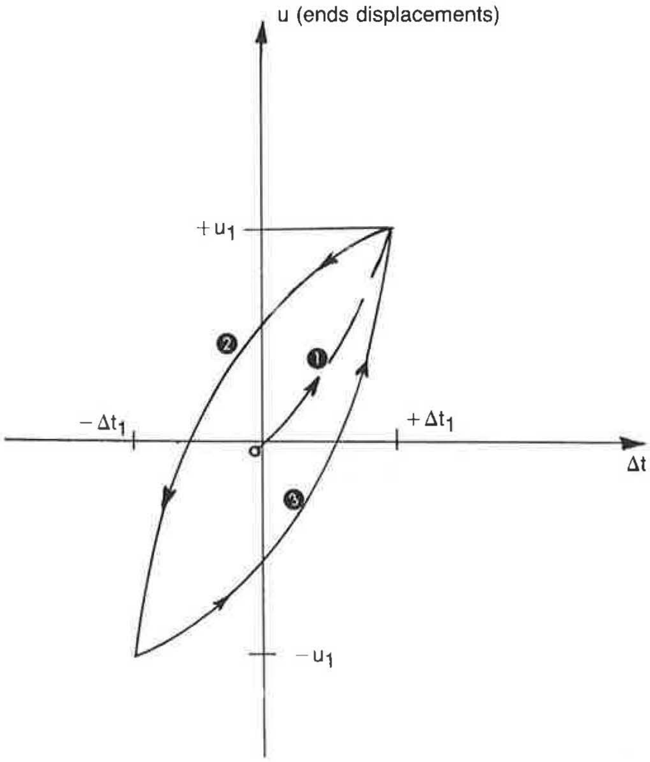
C = torque density per meter of track (kN),
α = rotation angle (rad), and
K = coefficient (kN/rad).

Taken into account is the linear part of the function $C = f(\alpha)$, before the horizontal part of the experimental C value. Torsional resistance of the fastenings is shown in Figure 7.

Specified ballast profiles give the track a sufficient transverse (and longitudinal, as shown before) resistance to avoid geometric defects and buckling. From the longitudinal standpoint, the ballast section must accommodate the longitudinal rail forces operative in the breathing lengths. The vertical resistance must be examined from the track modulus standpoint, not in terms of the friction coefficient. The most important part of the resistance is the lateral resistance, which withstands the lateral displacement of the track caused by transverse forces. The transverse resistance, sometimes characterized as τ_l, is represented in Figure 8 by the values of *kN/m*. When considering CWR track, two factors must be taken into account—nonconsolidated track and consolidated track. A value for the second factor is obtained after a minimum of 100 000 tonnes of traffic or dynamic stabilization. Values for the transverse resistance of two segments of rail are presented in Table 1. Note the difference between concrete and timber in weight and the difference between consolidated and nonconsolidated track, particularly in the timbered track.

The two curves in Figure 8 are different in value because the horizontal part of the simplified one must be considered as a maximum and in terms of rigidity for the first part of the graph. The real function, bounded to the friction coefficient of the track frame on the ballast, is simplified to allow a reasonable and theoretical approach. The track weight is essential for the value of the transverse resistance of the track.

Finally, control of track geometry imperfections is essential for acceptable track behavior: the lower the quality of the geometry, the more the track is prone to buckling. The am-



$+u_1$ and $-u_1$ provide limits of the expansion zone

FIGURE 6 Movement of CWR ends.

plitude of defects in track geometry, which increases with compression forces (the wavelength of the defect being unchanged), leads to the point of buckling. The relationship between lateral geometric defects and longitudinal compression forces is illustrated in Figure 9. As the defects increase, the lateral track resistance becomes more involved and reaches its limit at the point of buckling. At that moment the graph shows the horizontal zone of the lateral resistance of the track. A critical amplitude of the defect can be defined to cope with the maximum temperature variation between the stress-free temperature and the maximum temperature the track can reach before buckling.

THEORETICAL APPROACH TO TRACK STABILITY

Vertical Stability

It is assumed that the track is perfectly level when the stress-freeing operation is carried out. The occurrence of leveling defects takes place after a certain amount of traffic, in the form of subgrade or soil deformation.

A longitudinal defect is shown in Figure 10. For a compression force F the ballast reaction r becomes less than the normal weight of the track. This F value is calculated with the equilibrium equation of a track element. For each F value of longitudinal force, the maximum weight relief is determined by a critical wavelength of the defect, which tends to maximize its amplitude. When all calculations are made, this wavelength is

$$L = 8.88 \left(\frac{EJ}{F}\right)^{1/2}$$

where

J = rail vertical inertia,
E = Young's modulus, and
F = longitudinal compression force.

The ballast reaction for a meter is

$$r = \overline{\omega} - b\,\frac{F^2}{4EJ}$$

where $\overline{\omega}$ is the weight of the track (in kilonewtons per meter). Weight increases at the bottom of the wave and decreases at the top. Loss of track weight can be taken into account with this result to reduce the lateral resistance of the track. For the French National Railroads, assuming loss of track weight, the calculation for the maximum defect is

$$2\,b = \frac{L}{1{,}000}$$

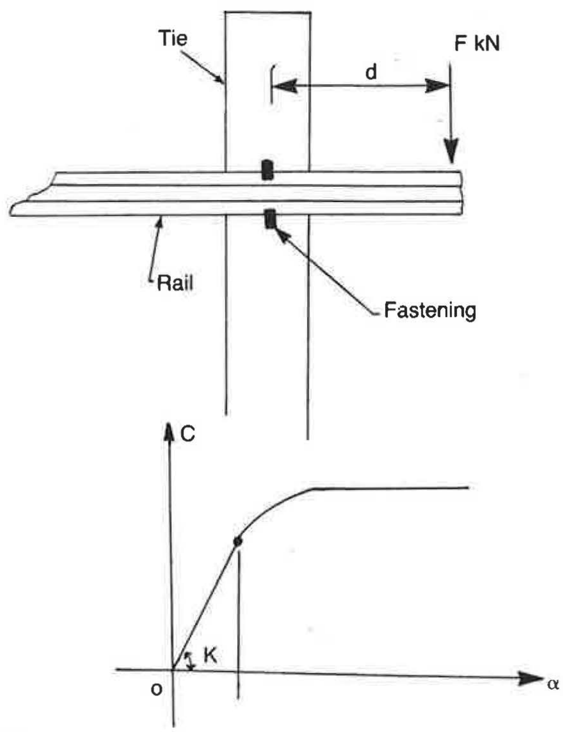
$$b = \frac{L}{2{,}000}$$

Transverse Stability

The essential factors that directly affect the transverse stability of the track are

- Rail inertia,
- Torsional resistance of the fastenings,
- Ballast profile resistance to the transverse forces in the linear part of its stiffness, and
- Track geometry quality.

Rail inertia is well known and need not be discussed here. The torsional resistance $C = K\alpha$, proportional to the rotation of the rail on the tie, is an experimental value. The graph of the transverse resistance of the ballast is taken in the part of linear displacements (function of the transverse force). It is also an experimental value, with two figures, in the case of consolidated track and nonconsolidated track. For example, if a track has no alignment defect and is perfectly straight, it is impossible to trigger buckling with normal climate conditions. The longitudinal forces ($F = ES\,\alpha \cdot \Delta t$) would be too moderate as far as the temperature variations are concerned between the stress-free temperature and the maximum rail temperature. An alignment defect at the stressing (or laying) operation often results in track buckling. The amplitude of the defect at the stress-free temperature increases with the temperature, and the wavelength remains unchanged. This increasing amplitude gradually absorbs the transverse resistance of the ballast, up to the buckling point. To carry out a



$C = K\alpha$

Units:

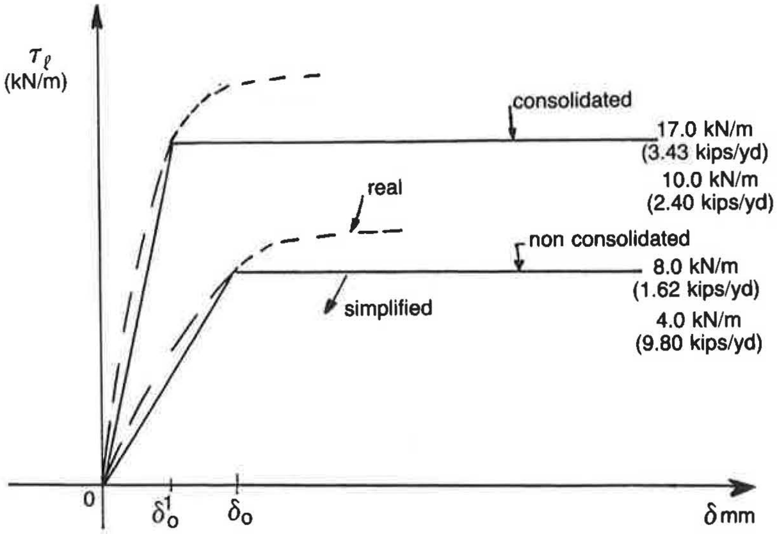
for C, $\frac{m \times kN}{m} = kN$

for K, $\frac{kN}{rad.}$

Examples of current figures:
- fastening on concrete ties: K = 100 kN
- fastenings on timber ties: K is situated between 45 kN and 150 kN

(These values for K are typical for SNCF fasteners.)

FIGURE 7 Torsional resistance.



Experimental figures for concrete and timber ties

Specified ballast profile for concrete ties

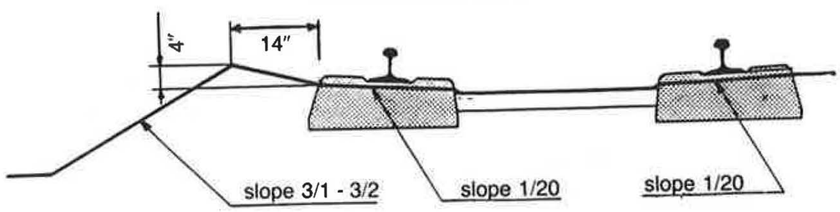


FIGURE 8 Transverse resistance of the track.

TABLE 1 TRANSVERSE RESISTANCE VALUES OF TWO SEGMENTS OF RAIL

TIE	RAIL	TRACK NON CONSOLIDATED (kN/m) kips/yard		CONSOLIDATED	
		BALLAST PROFILE STANDARD	BALLAST PROFILE REINFORCED	BALLAST PROFILE STANDARD	BALLAST PROFILE REINFORCED
TIMBER	110 lbs/y	(1,700) 0,34	(1,940) 0,39	(5,200) 1,03	(5,750) 1,14
CONCRETE	"	(3,500) 0,70	(3,920) 0,78	(5,900) 1,17	(8,500) 1,69
TIMBER	120 lbs/y	(1,770) 0,35	(2,010) 0,40	(5,200) 1,03	(5,750) 1,14
CONCRETE	"	(3,500) 0,70	(3,920) 0,78	(5,900) 1,17	(8,500) 1,69

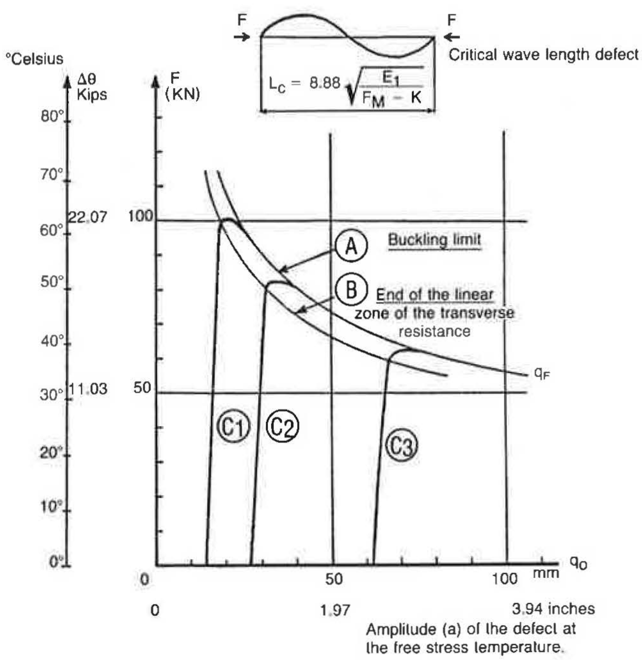


FIGURE 9 Relationship between lateral geometrical defects and longitudinal compression forces.

theoretical approach to the problem, a sinusoidal shape of the defect in a tangent track is considered.

The defect is defined by

$$Y = a \cos \omega x$$

$$\omega = \frac{2\pi}{L}$$

where L is the wavelength of the defect. It is possible to define the amplitude of the defect with a versine value, for example, a 10 m chord:

$$f = a\,(1 - \cos 5\omega)$$

The maximum value of the amplitude (a_{max}) of the defect at the stress-free temperature is

$$a_{max} \leq \frac{\tau_l - \dfrac{F}{R}}{EI\,\omega_{max}^4\left(1 + \dfrac{F\,\omega_{max}^2}{k' - EI\,\omega_{max}^4}\right)} \qquad (1)$$

This maximum function occurs with a critical wavelength L_{max}.

$$L_{max} = 8.88\left(\frac{EI}{F - K}\right)^{1/2}$$

$$\omega_{max} = \frac{2\pi}{8.88}\left(\frac{F - K}{EI}\right)^{1/2}$$

The formula in Equation 1 enables the maximum value of longitudinal forces or the maximum value of a defect to be found, according to the chosen assumptions. Previously, it was found that the track has a weight loss in the case of the track level defect. A normal value of this weight loss (i.e., loss of transverse resistance $\Delta_1\tau$) must be taken into account.

Finally, geometric defects of the rail, that is, manufacturing process defects of the rail at the rolling plant, and the welding process defects must be examined. They have an important influence on the real transverse resistance of the track. The lateral force used by the ballast profile to correct these defects reduces its total value. It is assumed that a rail geometric

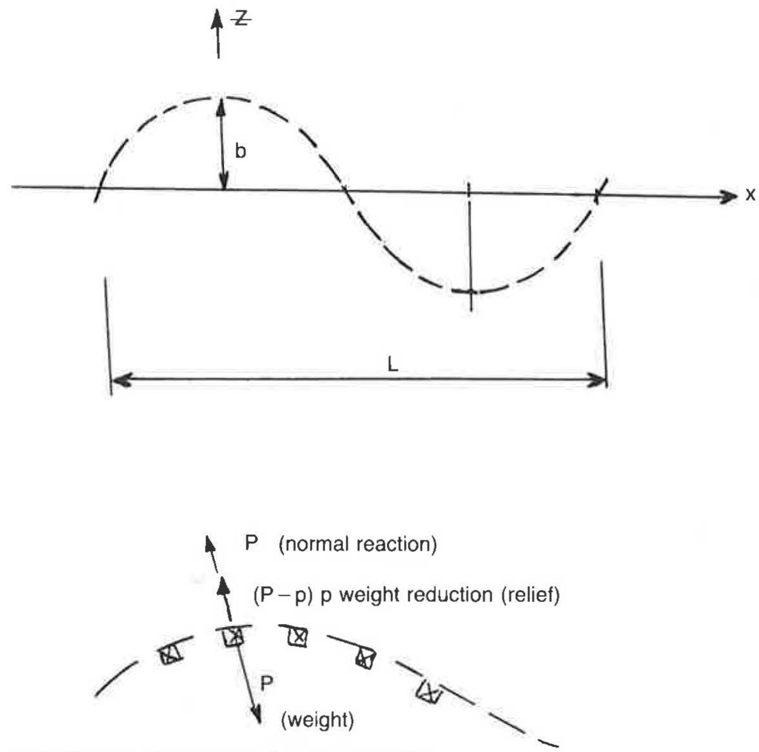


FIGURE 10 Longitudinal defect in CWR.

defect of 1/1,000 rad or a defect of an amplitude of 2.5 mm (1/10 in.) with a 10-m wavelength is the maximum value allowed in CWR track. The transverse resistance of the ballast layer is reduced as follows to take account of geometric defects of the rails:

$$\Delta_2 \tau = a_2 \omega_2^2 (EI\, \omega_2^2 + K)$$

where a_2 is the amplitude of the defect and ω_2 equals $2\pi/L_2$. A 20 percent reduction of the vibrations (0.80 coefficient) must also be taken into account. The final value is

$$\tau' = 0.8\, (\tau_1 - \Delta_1 \tau) - \Delta_2 \tau$$

Equation 1 when used for tangent tracks becomes

$$a_o \geq \frac{\tau_l' - \dfrac{F}{R}}{EI\, \omega_o^4 \left(1 + \dfrac{F\, \omega_o^2}{k' - EI\, \omega_o^4}\right)}$$

where

$\left(\tau_l' - \dfrac{F}{R}\right)$ = the final transverse resistance of the track,

τ_l = the total transverse resistance, taking into account all the reductions, and

$\dfrac{F}{R}$ = the transverse component of F in a curve with Rm radius.

Each unknown quantity can be calculated by taking into account assumptions for the others. For example, a minimum R (m) can be found in assuming a maximum alignment defect, a maximum F, and a maximum rail geometry defect. An equivalent approach can be carried out with graphs of the function

$$F_{max} \text{ (or } \Delta t) = f\,(Rm).$$

Track stability is shown in Figure 11. A curve can be obtained for each transverse resistance value, and with a specified radius of a curve, it is possible to learn the maximum value for F (i.e., Δt maximum). These curves are useful to study special locations in the CWR at which longitudinal forces might be different compared with a standard situation. Some cases involve force variations in the middle part of the CWR.

SITE CONDITION EFFECTS

CWR behavior was examined previously with some assets: total length of the CWR in a similar site condition—plain line, same sun exposure, no bridge, no switch, and no tunnel. Real conditions are slightly different, and the result causes disturbances in the standard behavior of the CWR. The track stability must withstand these disturbances, which are dangerous when they increase the compression stresses. For example, when a long bridge with an uninterrupted longitudinal beam is in the central part of the CWR, a maldistribution of the stresses along the CWR is noticed at the free end, that

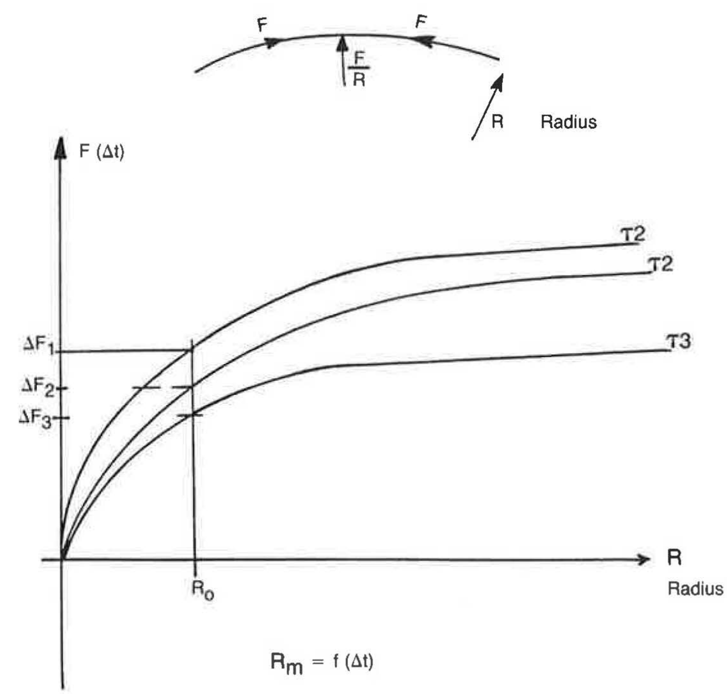


FIGURE 11 Track stability.

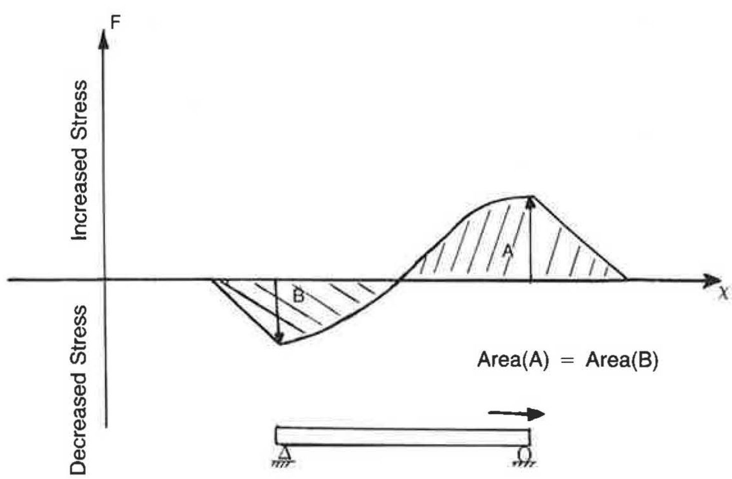


FIGURE 12 Compression stresses of CWR on a bridge.

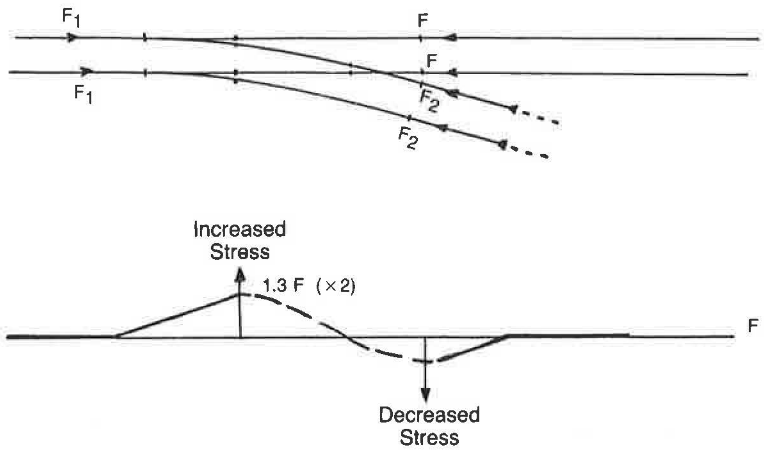


FIGURE 13 Compression stresses of CWR at a turnout.

is, stresses increase at the free end and decrease at the other. These increased stresses or (forces) are caused by the friction of the ballast on the bridge spans, which drags the track toward the free end. The distribution of stresses (*F*) is shown in Figure 12. This inclusion of bridges in CWR systems requires that both the radius of the track and the length of the spans be limited in order to keep the track in a state of balance. Peaks of longitudinal stresses are distributed, as they are in a breathing zone.

Other sites that affect the behavior of CWR include tunnels with welded rail throughout and turnouts with particular conditions, in which four stretches of rail become two stretches with increased compression forces in each of them. The compression stresses of a turnout in CWR are illustrated in Figure 13. The difference between the crossing heel with four rails and the toe-switch involves increased stresses at the heel of the stock-rail. The coefficient of overvalue is 1.30; the major part of the longitudinal force is blunted by flexion of the supports and friction of the rail on the bearers before the heel of the switch rail. The track stability must withstand this 30 percent increase in stresses (forces).

CONCLUSIONS

Mathematical development and all experiments done to verify the results of the theoretical approach are defined in this paper. All this work was done by French National Railroad engineers, who were primarily the first developers and who transferred the results in practical applications to the French rail network. These applications are possible with specific procedures performed in accordance with local situations and climate conditions; the first procedure applies to laying and stressing. The CWR system gives an accurate and reliable solution to problems of jointed track that relate to buckling and maintenance costs. The theoretical method to find the limit of track stability and its application, which takes into account every potential defect at the same moment (e.g., track geometry, levelings, and rail quality), gives to the track a state of balance, the high level of security coefficient it needs. Strict application of the CWR maintenance rules by all staff on the tracks, however, is the guarantee of the system's excellent reliability.

Dynamic Buckling of Continuous Welded Rail Track: Theory, Tests, and Safety Concepts

A. KISH AND G. SAMAVEDAM

A versatile, dynamic buckling model that can be used on a personal computer is presented. The model accounts for vehicle load influences and nonlinearities in track resistance, hitherto ignored in the literature. These influences are shown to be important in the accurate predictions of buckling response and hence in buckling safety considerations. The model also computes the energy required to buckle the track and thus indicates the levels of safety at given rail temperatures. On the basis of the energy and the upper and lower buckling temperatures derived from the model, rational buckling safety criteria have been developed. Results of controlled full-scale dynamic buckling tests conducted on tangent, 5-, and 7.5-degree continuous welded rail track are presented and correlated with theoretical predictions from the model on buckling temperatures, forces, and safety limits.

Thermal buckling of continuous welded rail (CWR) track is an important problem facing the safe operation of railroads in the United States. Increased utilization of CWR and recent trends toward higher speeds and heavier axle loads are expected to exacerbate this problem. In an effort to improve the safety of CWR track, analytical and experimental investigations have been conducted by the Transportation Systems Center (TSC) in support of the safety mission of FRA. Investigations of CWR track buckling under thermally induced forces and vehicle loads are described in this paper.

The TSC approach to the solution of the buckling problem consists of

- Developing a rigorous model based on fundamental principles of structural mechanics that accounts for all significant parameters,
- Validating the model by controlled full-scale field tests, and
- Developing rational safety criteria for use by the industry.

Static buckling is defined as the buckling of long CWR tracks caused by thermal load alone with no interaction from vehicles. Most of the published literature deals with this type of buckling. In contrast, dynamic buckling, which is more relevant to the industry, is defined as the instability of CWR track under moving vehicles in the presence of thermal loads. The dynamic buckling aspects of CWR track are the focus of this paper.

A. Kish, Transportation Systems Center, DTS-76, 55 Broadway, Cambridge, Mass. 02142. G. Samavedam, Foster-Miller, Inc., 350 Second Avenue, Waltham, Mass. 02254.

REVIEW OF STATIC BUCKLING

Before the development of dynamic buckling theory, TSC conducted theoretical studies and field tests of static buckling. The studies were based on early work by Kerr (*1*) and Samavedam (*2*). Kerr's work defined the basic large deflection analysis required in the thermal buckling problem for tangent tracks. Samavedam generalized the various nonlinearities in the input parameters and proposed the first rigorous analysis for curved tracks.

In 1982 Kish et al. (*3*) conducted the first series of static buckling tests on U.S. mainline tangent and 5-degree-curve track to better define the buckling response mechanism and characteristics. A significant number of theoretical parametric studies on static buckling have also been conducted (*4*). These and subsequent research efforts have clearly identified the need for a more comprehensive analytic model that incorporates several nonlinear parameters and dynamic effects and for rational buckling safety criteria.

Recent advances in the analytic modeling of the dynamic buckling behavior of CWR track, some relevant validation tests, and proposed safety criteria that may provide a basis for rational guidelines for buckling prevention are presented here.

LIMITATIONS OF EXISTING THEORIES

Before 1985, all known theories published in the United States and elsewhere had three major deficiencies:

- Inadequate representation of lateral resistance,
- Lack of vehicle load effects, and
- No rational criteria for CWR buckling safety.

In 1985, Kish et al. (*5*) published the first work on dynamic buckling, which covered various buckling mechanisms arising from vehicle loads. This work recently has been extended to rectify the deficiencies listed above.

For further development, appropriate terminology must be introduced. The lateral buckling response can be expressed in the form of a relationship between the maximum lateral track displacement and the temperature increase over the force-free or neutral temperature, as shown in Figure 1.

At point B, the structure becomes unstable, even under an infinitesimal disturbance. $T_{B,\max}$ is the upper buckling temperature, the maximum temperature limit before the track buckles. The track could also buckle at $T_{B,\min}$ from its stable

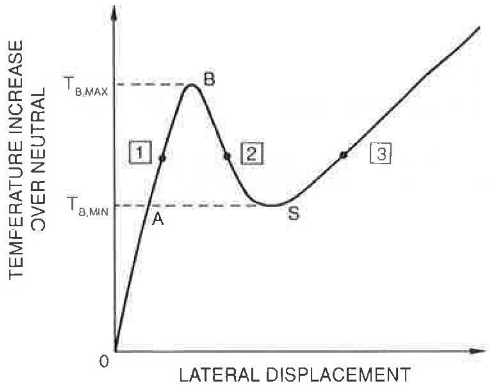


FIGURE 1 Typical buckling response.

equilibrium position A to S, if given sufficient external disturbance, such as forces developed by a moving train. $T_{B,\min}$ is defined as the lower buckling temperature, which, as seen later, may or may not equal a safe allowable temperature.

Lateral Resistance Characteristic

TSC performed a large number of track lateral resistance evaluation tests. Both panel pull and single-tie push tests (STPTs) were executed and the results were correlated. As described by the authors in another paper in this Record, a special portable test fixture for individual tie resistance evaluation has been developed. Typical results for U.S. track are shown in Figure 2. The results identify two salient points, F_p and F_L, which are the peak resistance and the limiting resistance. Except in the case of extremely weak tracks, the resistance has a "softening" characteristic after reaching the peak value. The full characteristic is important in the buckling analysis because at temperatures equal to or greater than the lower buckling temperature ($T_{B,\min}$) the resulting deflections are large. Many existing works considered only the peak resistance in the determination of the buckling response and significantly overestimated the values of $T_{B,\min}$, the implications of which will be discussed later.

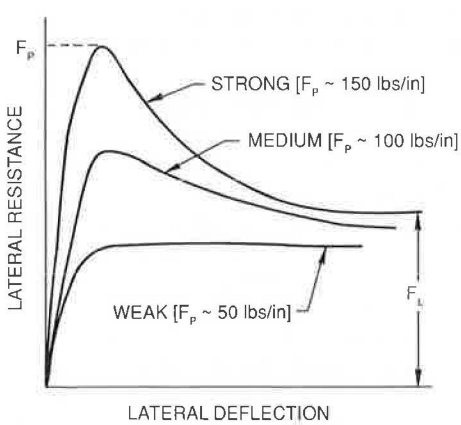


FIGURE 2 Typical single-tie push test results.

Vehicle Load Effects

Research conducted by the French National Railway (SNCF) indicates that most track buckling is caused by vehicle passage (*6*). According to a survey by the Association of American Railroads, 68 percent of derailment-inducing buckling occurred under the train consist; 6 percent occurred in front of the locomotive (*7*). Tests conducted by the Hungarian State Railways indicated that vehicle traffic can reduce the buckling strength by 20 to 30 percent (*8,9*). These data and the results of testing by TSC, which will be presented later, indicate the importance of including vehicle effects in buckling analyses.

Work by Kish et al. (*5*) contains a review of literature on vehicle effects published before 1985. The following mechanisms were identified to be important in constructing an appropriate dynamic buckling theory:

1. Uplift of the track due to precession/recession and central bending waves can reduce the lateral resistance and, hence, buckling strength.
2. Lateral forces generated on the track due to wheel/rail interaction (especially in the presence of lateral imperfections), in combination with many passes of the vehicle, can increase the size of the imperfection and therefore reduce the buckling strength.
3. Braking, traction, and flanging forces can also increase compressive forces and hence reduce buckling strength.
4. Track vibration caused by passage of a vehicle can cause loss of lateral ballast resistance.

Detailed calculations on Mechanism 1 are presented in work by Kish et al. (*5*). The central bending wave for long cars and the precession wave for locomotives are generally important in buckling evaluation, as shown in Figure 3. The work presented in this paper accounts for the loss of lateral resistance caused by the uplift of the track, allowing for self-weight of the track. The uplift mechanism has been previously identified as one of the principal causes of buckling by European researchers, including Eisenmann (*10*). An experimental proof of the effect of this mechanism will be provided later.

The effect of the ratio of truck lateral to vertical loads (L/V), as implied in Mechanism 2, was considered by Kish et al., who concluded that L/V becomes critical if it exceeds the friction coefficient between tie and ballast (*5*). The same conclusion was reached earlier by SNCF (*6*). Limited studies have been performed to date on Mechanism 3, and no work has been done in the United States on Mechanism 4. The TSC approach is to combine the influence of those dynamic factors into a dynamic margin of safety, which will be discussed later.

Basis for Buckling Safety Criteria

Previous works recommend the lower buckling temperature as the safe allowable limit for CWR track. As shown later, this approach can be conservative in some cases. An optimum safe allowable temperature must therefore be established. This can be done through energy considerations presented here.

At the upper buckling temperature, the external energy required to buckle the track is zero. This temperature cannot practically be reached without buckling the track under dy-

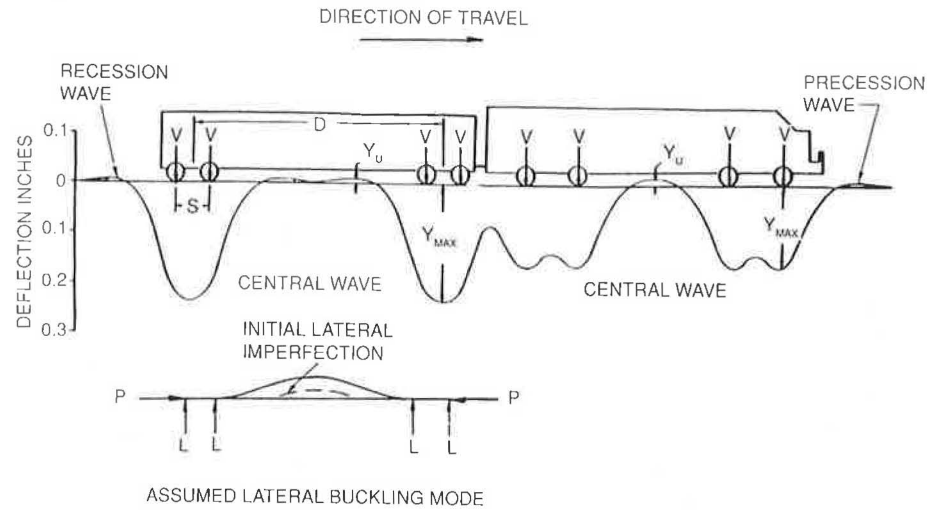


FIGURE 3 Typical track deflections caused by GP38-2 locomotive and hopper car.

namic conditions because trains always cause some finite disturbance. Nevertheless, the energy required to buckle the track at the lower buckling temperature may be considerably greater than that generated by moving trains. The track's buckling potential at different rail temperatures for given parameters can be evaluated through calculation of the energy required for buckling. As shown later, energy calculations provide a rational basis for defining operational temperatures with a given level of safety.

TSC BUCKLING MODEL

A buckling model has been developed by TSC using the differential equations described in the next section. It has the following features:

- It applies to tangent and curved tracks.
- Lateral alignment defects are included.
- It accounts for any nonlinearity in the lateral resistance, including the softening behavior referred to previously. The individual contributions of tie bottom, crib, and shoulder to the lateral resistance become important in the model.
- Linear or nonlinear longitudinal resistance can be incorporated.
- It considers vehicle load influences and accounts for lateral resistance loss or variation under the cars. Car parameters such as truck center spacing and wheel load are included, as are track modulus and tie-ballast friction coefficient.
- It calculates the external energy required for an explosive (sudden) buckling and thus indicates the potential risk of buckling at a given rise in rail temperature.
- It can be run on a personal computer (PC), with simple user-friendly inputs. It can be operated as an expert system, requiring no knowledge of the theoretical equations involved. The program has default options and automatically assumes missing input if not provided by the operator.
- The output can be in the form of buckling response curves, with printout of upper and lower buckling temperatures, energy, and risk factors.
- Within the limitations of the physical assumptions, the model is extremely accurate, relying on differential equations and fast converging Fourier series solution.

Buckling Response Determination

A basic formulation for tangent track has been provided by Samavedam et al. (*11*). Here, the formulation for curved track not presented in earlier work is given. The following assumptions are made:

- The two rails can be combined into a single beam of known cross-sectional area A and flexural rigidity EI.
- The torsional stiffness in rail-tie fasteners may be neglected, which is reasonable for the majority of wood-tie tracks with tie plate–cut spike construction in the United States.
- The buckled zone with lateral displacements is confined to a finite length. This has been confirmed by tests (*12,13*). The longitudinal resistance offered by the ballast to the longitudinal movement of the rail beam can be neglected in the buckled zone, which will simplify the solution of the resulting differential equations.
- The adjoining zone experiences only longitudinal movement, and the rail force at infinity is $P_\infty = AE\alpha T$, where T is the increase in temperature over the stress-free temperature and α is the rail steel's coefficient of thermal expansion. The longitudinal resistance can be linear or nonlinear. As shown by Samavedam (*2*), there are no theoretical difficulties in handling the nonlinearity in the resistance. For simplicity, linear idealization will be used here because it appears to be adequate, on the basis of recent field test data.

The lateral resistance is idealized as follows.

Partial "softening" lateral resistance:

$$F[w(x)] = F_p[k + (1 - k)\exp(-\mu_2 w)] \tag{1.1}$$

Full "softening" lateral resistance:

$$F[w(x)] = F_p[1 - \exp(-\mu_1 w)]\left\{k + (1 - k) \times \exp\left[-\mu_2\left(w - \frac{4}{\mu_1}\right)\right]\right\} \quad (1.2)$$

where

F_p = value of the peak lateral resistance,
k = ratio of reduced to peak lateral resistance,
μ_1 and μ_2 = stiffness parameters that define the initial and softening behavior of the assumed lateral resistance function, and
w = lateral or radial track deflection.

Examples of the idealizations are shown in Figure 4.

For the case in which vehicle loading is present, the peak resistance (F_p) is a function of the longitudinal distance along the track:

$$\overline{F}_p[w(x)] = \begin{cases} [F_p - \mu Q] \text{ for uplift} \\ [F_p + \mu R_v(x)] \text{ otherwise} \end{cases} \quad (2)$$

where

F_p = peak value of static lateral resistance,
μ = tie to ballast coefficient of friction,
Q = self-weight of the entire track, and
$R_v(x)$ = vertical deflection profile produced by the vehicle wheel loads on the track.

The vertical deflection profile can be calculated from the classical theory for beams on elastic foundation. Uplift occurs when the sum of the vertical deflection and the self-weight of the track is less than zero ($[Q + R_v(x)] < 0$).

Governing Equations for Curved Track Analysis

For the geometry and coordinate system shown in Figure 5, the governing differential equation in the buckled zone ($0 \leq \Theta \leq \phi$) for curved track is given by Samavedam (*2*) as

$$\frac{EI_{zz}}{R^4}\frac{d^4w}{d\Theta^4} + \frac{\overline{P}}{R^2}\frac{d^2w}{d\Theta^2} = -F[w(\Theta)] + \frac{\overline{P}}{R} - \frac{\overline{P}}{R^2}\frac{d^2w_o}{d\Theta^2} \quad (3)$$

where

EI_{zz} = flexural rigidity of both rails in the lateral plane,
$\overline{P}$ = rail compressive force,
w = lateral or radial displacement,
w_o = initial misalignment, and
F = the lateral resistance.

The Fourier method originally given by Samavedam (*2*) is used for the solution of Equation 3:

$$w(\Theta) = \sum_{m=1,3,5\ldots}^{\infty} A_m \cos\left(\frac{m\pi\Theta}{2\phi}\right) \quad (4.1)$$

$$\frac{\overline{P}}{R^2}\frac{d^2w_o}{dx^2} = \sum_{m=1,3,5\ldots}^{\infty} b_m \cos\left(\frac{m\pi\Theta}{2\phi}\right) \quad (4.2)$$

$$F[w(x)] = \sum_{m=1,3,5\ldots}^{\infty} a_m \cos\left(\frac{m\pi\Theta}{2\phi}\right) \quad (4.3)$$

$$\frac{\overline{P}}{R} = \sum_{m=1,3,5\ldots}^{\infty} c_m \cos\left(\frac{m\pi\Theta}{2\phi}\right) \quad (4.4)$$

Using the differential equation, it can be shown that

$$A_m = \frac{-\left[\left(a_m - \frac{\overline{P}}{R}c_m\right) + \frac{\overline{P}}{R^2}b_m\right]}{\left[\frac{EI_{zz}}{R^4}\left(\frac{m\pi}{2\phi}\right)^4 - \frac{\overline{P}}{R^2}\left(\frac{m\pi}{2\phi}\right)^2\right]} \quad (5)$$

The Fourier coefficient that accounts for the effects of lateral resistance in the curved track case is derived from

$$a_m = \frac{2}{\phi}\int_0^{\phi} F[w(\Theta)] \cos\left(\frac{m\pi\Theta}{2\phi}\right) d\Theta \quad (6.1)$$

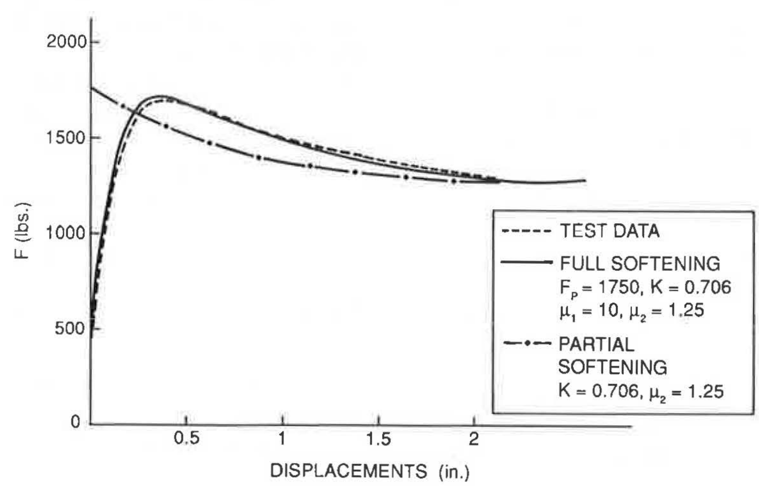


FIGURE 4 Lateral resistance test data and idealizations.

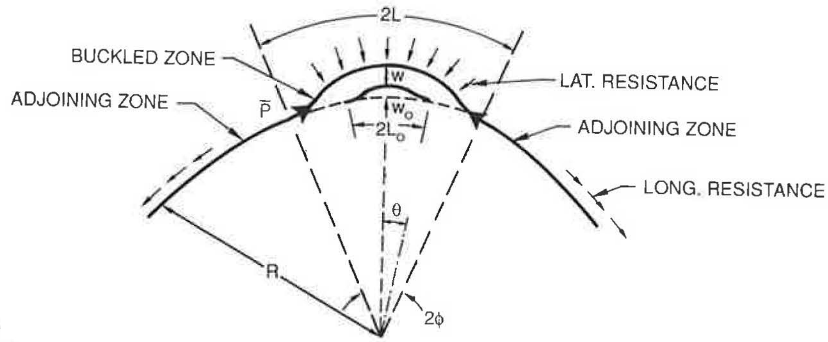


FIGURE 5 Geometry and coordinates for curved track.

This integral is evaluated numerically by using Filon's method.

The Fourier coefficient that accounts for the effect of imperfection is obtained from the following integral:

$$b_m = \frac{2}{\phi}\int_0^{\phi^*} \frac{d^2 w_o}{d\Theta^2} \cos\left(\frac{m\pi\Theta}{2\phi}\right) d\Theta \tag{6.2}$$

where

$$\phi^* = \begin{cases} \phi & \text{if } \phi \le \phi_o \\ \phi_o & \text{if } \phi > \phi_o \end{cases}$$

A quartic imperfection shape is assumed (although the analysis is capable of dealing with any form of imperfection) as

$$w(\Theta) = \delta_o\left[1 - 2\left(\frac{\Theta}{\phi_o}\right)^2 + \left(\frac{\Theta}{\phi_o}\right)^4\right] \tag{7.1}$$

where δ_o is the "offset" or the amplitude and $2R\phi_o = 2L_o$ is the length over which the imperfection occurs. Thus, evaluation of the integral for b_m results in the following:

If $\phi \le \phi_o$,

$$\frac{b_m}{R^2} = -\frac{16\,\delta_o}{m\pi L_o^2}\left\{1 - 3\left(\frac{L}{L_o}\right)^2 \times \left[1 - 2\left(\frac{2}{m\pi}\right)^2\right]\right\} \sin\left(\frac{m\pi}{2}\right) \tag{7.2}$$

If $\phi > \phi_o$,

$$\frac{b_m}{R^2} = -\frac{16\,\delta_o}{m\pi L_o^2}\left\{-6\left(\frac{L}{L_o}\right)\left(\frac{2}{m\pi}\right)\cos\left(\frac{m\pi L_o}{2L}\right) + 2\left[-1 + 3\left(\frac{2L}{m\pi L_o}\right)^2\right]\sin\left(\frac{m\pi L_o}{2L}\right)\right] \tag{7.3}$$

Note that $\phi = L/R$ and $\phi_o = L_o/R$.

The remaining Fourier coefficient is

$$c_m = \frac{2}{\phi}\int_0^{\phi}\cos\left(\frac{m\pi\Theta}{2\phi}\right) d\Theta = \frac{4}{m\pi}\sin\left(\frac{m\pi}{2}\right) \tag{7.4}$$

The differential equation of longitudinal equilibrium that applies to the adjoining region ($\Theta > \phi$) and assuming proportional longitudinal resistance is

$$\frac{AE}{R^2}\frac{d^2U}{d\Theta^2} = K_f U \tag{8.1}$$

where U is the longitudinal or tangential displacement and K_f is the longitudinal stiffness. The general solution to this equation is

$$U(\Theta) = C_3 e^{+R\Psi\Theta} + C_4 e^{-R\Psi\Theta} \tag{8.2}$$

where $\Psi^2 = K_f/AE$. The temperature equation for curved track analysis is derived from the following boundary conditions:

$$\frac{U(\phi)}{R} = -\frac{\bar{P}\phi}{AE} - Z + \alpha T\phi \tag{8.3}$$

$$\frac{U'(\phi)}{R} = -\frac{\bar{P}}{AE} + \alpha T \tag{8.4}$$

The appropriate boundary conditions must be substituted into the solution, and it must be noted that $L = R\phi$. Solving for temperature results in the following expression:

$$T = \frac{\bar{P}}{AE\alpha} + \frac{ZR\Psi}{\alpha(1 + \Psi L)} \tag{9.1}$$

where

$$ZR = \int_0^{\phi}\left(\frac{w}{R} + \frac{w'^2}{2\,R^2} + \frac{w' w_o'}{R^2}\right) R\,d\Theta \tag{9.2}$$

$$ZR = \sum_{m=1,3,5\ldots}^{\infty}\left[\frac{2L}{m\pi R^2}A_m \sin\left(\frac{m\pi}{2}\right) + \left(\frac{m\pi}{2}\right)^2\frac{A_m^2}{4L} - \frac{A_m B_{mL}}{2\,R^2}\right] \tag{10}$$

Energy Required for Buckling

The prebuckling state is represented by Position 1 in Figure 1, and the postbuckling unstable branch is represented by Position 2. It is assumed that if the track can be brought into Position 2, it will automatically move to Position 3.

The following factors are defined:

V_1 = strain energy in the rails at stable equilibrium Position 1,
V_2 = strain energy in the rails at unstable equilibrium Position 2,
W = work done against resistances by moving track from Position 1 to Position 2, and
Ω = energy required to move track from Position 1 to Position 2.

By an energy balance

$$\Omega = (V_2 - V_1) + W \tag{11}$$

The strain energy components are given by the following integrals:

$$V_1 = \frac{1}{2}\int_0^\infty \frac{P_\infty^2}{AE}\,dx \tag{12}$$

where $P_\infty = -AE\alpha T$. Here, for simplicity, the energy caused by bending in the prebuckling state is neglected:

$$V_2 = \frac{1}{2}\int_0^\infty \frac{P^2}{AE}\,dx + \frac{EI_{zz}}{2}\int_0^\infty \left(\frac{d^2w}{dx^2}\right)^2 dx \tag{13}$$

In the curved track case, the longitudinal force distribution becomes

$$P = \begin{cases} \overline{P} & \text{for } 0 \le \Theta \le \phi \\ AE\left(\dfrac{1}{R}\dfrac{du}{d\Theta} - \alpha T\right) & \text{for } \Theta > \phi \end{cases} \tag{14}$$

The work components are given by the following integrals:

$$W_1 = \int_0^\infty \int_0^{w(x)} F[w(x)]dw\cdot dx \tag{15}$$

$$W_2 = \int_0^\infty \int_0^{u(x)} f[u(x)]du\cdot dx \tag{16}$$

Thus, the total work done against ballast resistance (lateral and longitudinal) is

$$W = W_1 + W_2 \tag{17}$$

The difference in strain energy is calculated from the following equation:

$$V_2 - V_1 = \frac{1}{2}\int_0^\infty \frac{P^2 - P_\infty^2}{AE}\,dx + \frac{EI_{zz}}{2}\int_0^\infty \left(\frac{d^2w}{dx^2}\right)^2 dx \tag{18}$$

This equation shows that the total strain energy is the sum of two components: one caused by compressive axial force and the other caused by beam bending. The evaluation of these integrals is performed with the aid of the Fourier analysis. Under the assumption of proportional longitudinal resistance, the difference in strain energy can be expressed in a "closed form":

$$V_2 - V_1 = \frac{AE}{2}\left\{\frac{\overline{P}}{AE}\left[\frac{\overline{P}}{AE}\left(L + \frac{1}{2\Psi}\right) + \frac{\alpha T}{\Psi}\right] - (\alpha T)^2\left(L + \frac{3}{2\Psi}\right)\right\} + \frac{EI_{zz}}{64\,L^3}\sum_{m=1,3,5,\ldots}^{\infty}(m\pi)^4 A_m^2 \tag{19}$$

The work done against lateral resistance can be evaluated from Equation 15 once the lateral resistance function is expressed mathematically. For the partial softening lateral resistance characteristics considered in the Fourier analysis section, the work done against lateral resistance is

$$W_1 = 2\int_0^L \overline{F}_p(x)\left[kw(x) + \left\{\frac{(1-k)}{\mu_2}1 - \exp[-\mu_2 w(x)]\right\}\right]dx \tag{20.1}$$

Full softening lateral resistance is

$$W_1 = 2\int_0^L \overline{F}_p(x)\left(kw(x) + \frac{k}{\mu_2}\{1 - \exp[-\mu w_1(x)]\}dx + \frac{(1-k)}{\mu_2}\exp\frac{4\mu_2}{\mu_1}\{1 - \exp[-\mu_2 w(x)]\} - \frac{(1-k)}{\mu_1+\mu_2}\exp\frac{4\mu_2}{\mu_1}\{1 - \exp[-(\mu_1+\mu_2)w(x)\}\right)dx \tag{20.2}$$

This integral is evaluated numerically.

The work done against longitudinal resistance,

$$W_2 = \frac{K_f}{4\,\Psi^3}\left(\frac{\overline{P}}{AE} - \alpha T\right)^2 \tag{21}$$

Illustrative Numerical Examples

Effect of Softening Lateral Resistance

The dynamic buckling response of 7.5-degree CWR curved track with both constant and softening lateral resistance characteristics is shown in Figure 6. The constant resistance idealization significantly overestimates the lower buckling temperature (77°F) compared with the softening characteristic (50°F). The buckling responses are also significantly different.

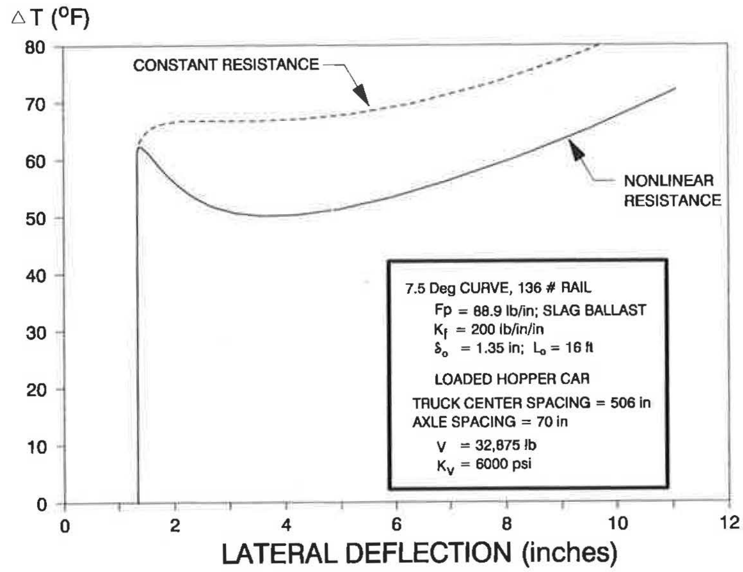


FIGURE 6 Influence of constant versus nonlinear resistance on buckling response.

Effect of Vehicle Loads

The theoretical buckling response of the 7.5-degree CWR curved track under hopper car loads is shown in Figure 7. The results for the static case without the vehicle are also shown. Because of the vehicle influence, the upper buckling temperature is reduced from 75° to 62°F.

Energy Required for Buckling

Figures 8 and 9 show the theoretical external energy required to buckle the tangent and 5-degree curved track with assumed parameters. This energy is clearly zero at the upper buckling temperature; hence, the track will buckle at this temperature. Buckling at the lower temperature requires a finite amount of energy. The energy required to buckle the track drops significantly with increased curvature and with line defects. The figures also indicate a rapid decrease in energy required with an increase in rail temperature above the lower buckling temperature.

BUCKLING SAFETY CONCEPTS

In order to assess buckling safety, temperature-deflection and temperature-buckling energy relationships from the TSC dynamic buckling model are required. Buckling can be "explosive" (snap-through) or "progressive" (gradual displacements). For explosive buckling, distinct upper and lower buckling temperatures are identified (see points $T_{B,\max}$ and $T_{B,\min}$ in Figure 1). For progressive buckling, these two points coalesce at an inflection point (a "knee" on the curve). This knee can be construed to be a progressive buckling temperature (T_p), because beyond this value larger displacements occur.

Margin of Safety Definition and Buckling Response Classification

As discussed previously, buckling can occur at any temperature between $T_{B,\max}$ and $T_{B,\min}$, depending on the energy imparted to the track by the moving train. Defining $\Delta = T_{B,\max} - T_{B,\min}$, it can be shown that the buckling energy increases as Δ increases; hence, Δ can be construed as a margin of safety against buckling. Using this definition, the buckling response characteristics can be classified into three cases as shown in Figure 10:

- Case I represents tracks exhibiting a buckling response for which $\Delta > 20$°F,
- Case II represents tracks exhibiting a buckling response for which 20°F $> \Delta > 0$, and
- Case III represents tracks exhibiting a progressive buckling response, $\Delta = 0$.

Figure 11 shows specific examples of these respective characteristics, including the energy required for buckling at $T_{B,\min}$ ($E_{\max}$) and the temperature above $T_{B,\min}$ corresponding to the 50 percent $E_{\max}$. For the example shown, it takes four times the energy to buckle at $T_{B,\min}$ for Case I than for Case II. This becomes important in defining required levels of safety based on low versus moderate risks of buckling potential.

Levels of Safety

Based on previous discussions of buckling strength characteristics, analytic considerations, dynamic buckling tests, and railroad industry response, Figure 12 summarizes buckling safety concepts based on two levels of safety. These levels of safety have been devised to provide a minimum (low) risk

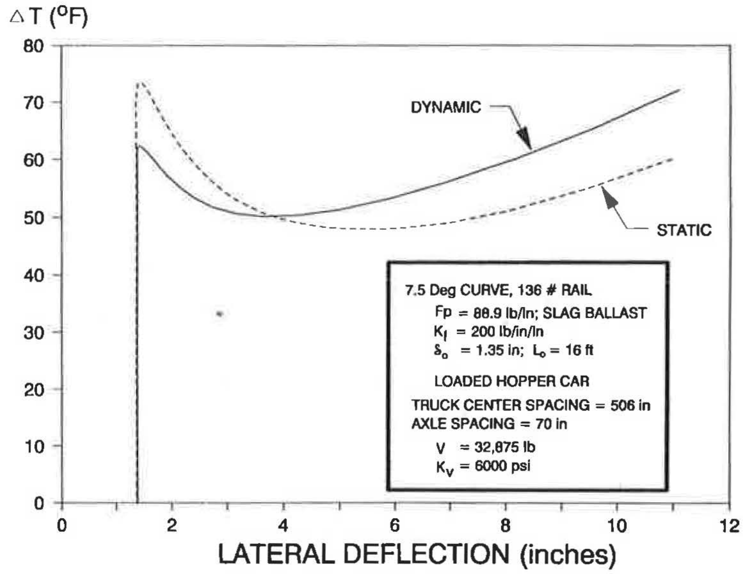


FIGURE 7 Influence of vehicle load on buckling response.

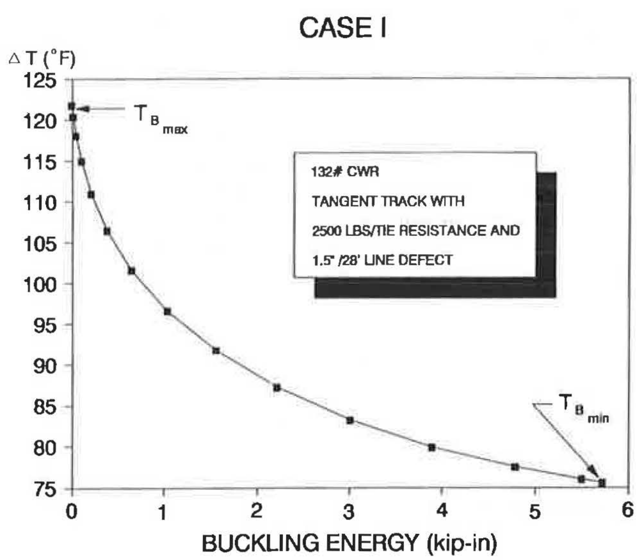


FIGURE 8 Buckling energy variation with temperature (tangent track).

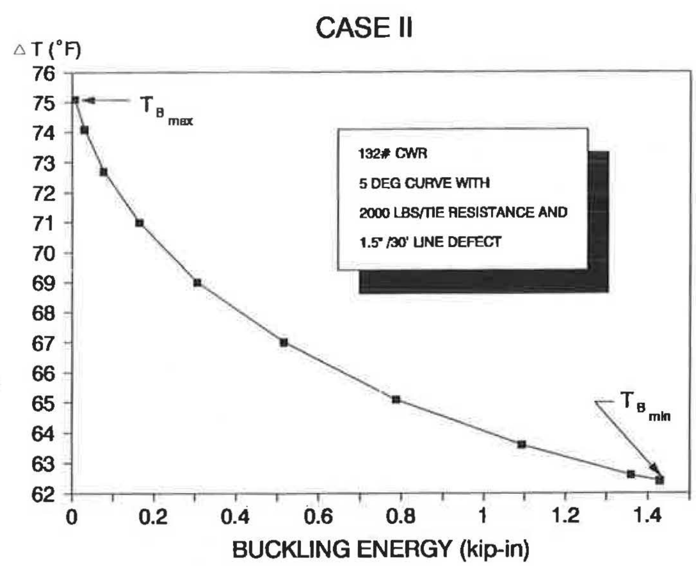


FIGURE 9 Buckling energy variation with temperature (curved track).

buckling potential and a marginal (moderate) risk buckling potential as illustrated in Figure 13:

- Level 1 safety (low-risk buckling potential) is based on $T_{B,min}$, $T_{B,min}$-20°F and T_p-20°F for Cases I, II, and III, respectively, for allowable temperature increase T_{ALL}, above neutral. The $T_{B,min}$ limit for Case I is justified by the typically high buckling energies at this temperature and by the fact that the actual $T_{B,min}$ values for Case I tracks tend to be higher than attained in most operating environments in the United States. The $T_{B,max}$-20°F limit is based on the moderately low buckling energies associated with Case II type tracks. The rationale for the 20°F safety margin is the need to account for some of the dynamic effects not included in the analysis. These include braking and traction forces, truck hunting forces, impact loads, and vibration-induced loss of track resistance. This 20°F safety margin also has some experimental basis, as shown in the next section. The T_p-20°F limit for tracks with progressive characteristics (Case III) is based on the relatively small lateral displacements associated with this temperature, a requirement to limit misalignment growth and lateral deflection to small values, and test results indicating that initiation of misalignment growth tends to occur approximately 20°F below the T_p value.

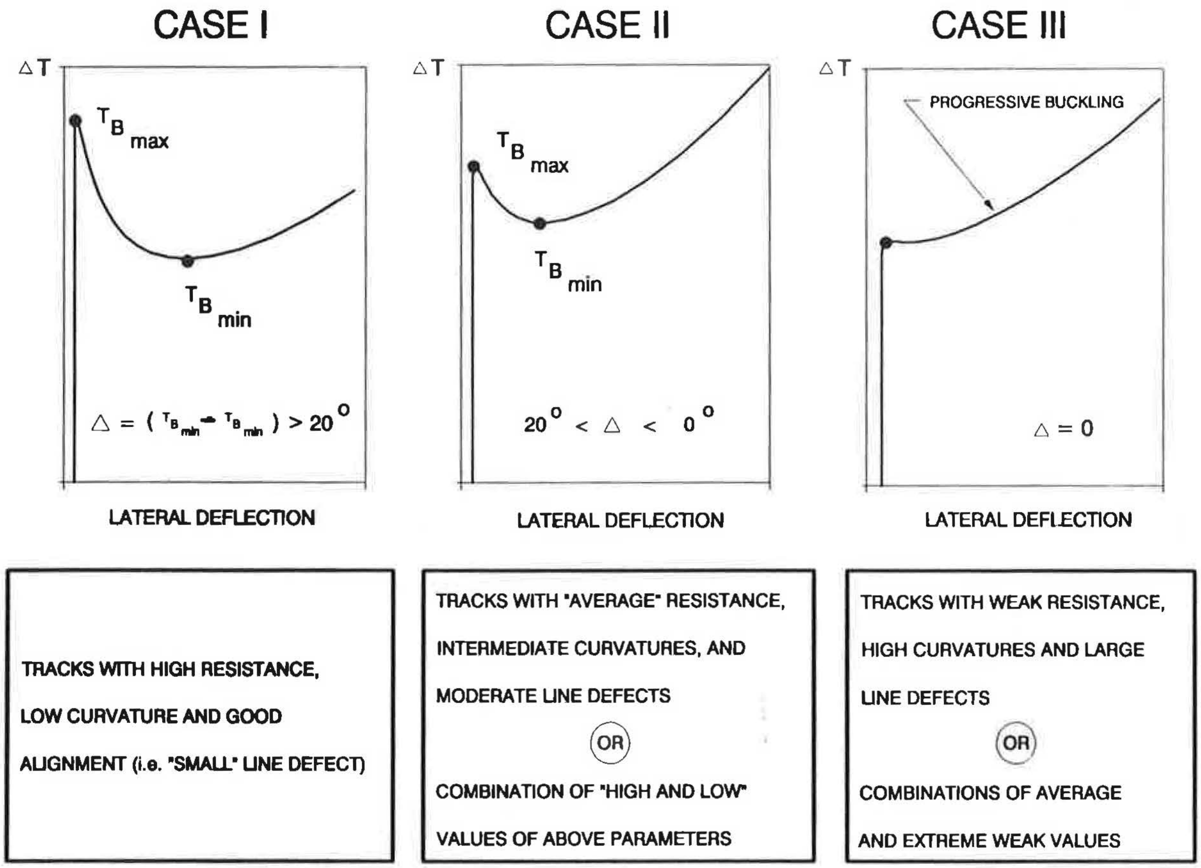


FIGURE 10 Classification of buckling characteristics.

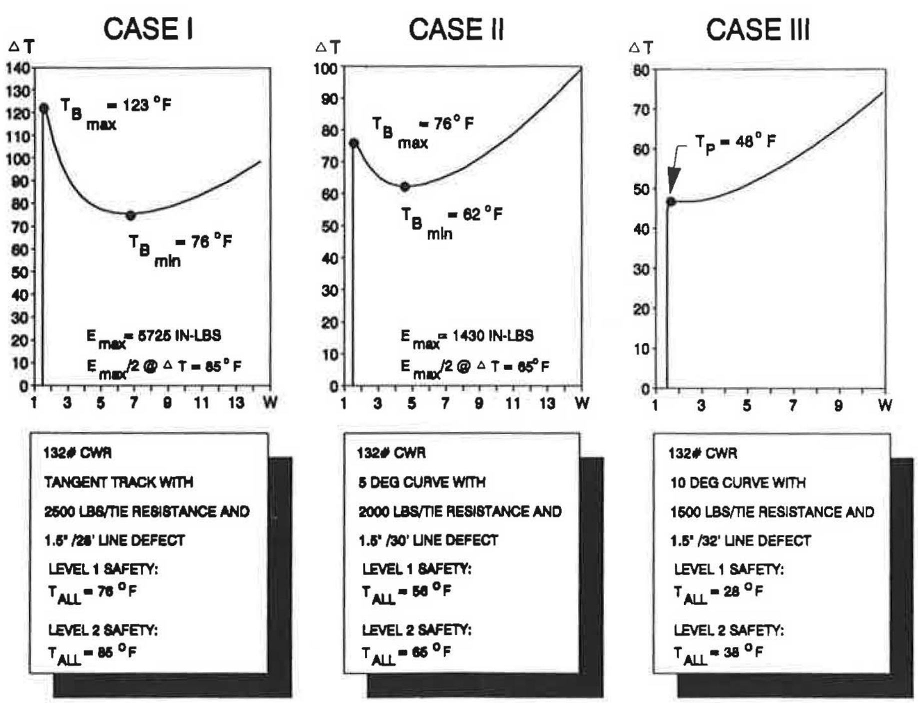


FIGURE 11 Typical buckling response examples.

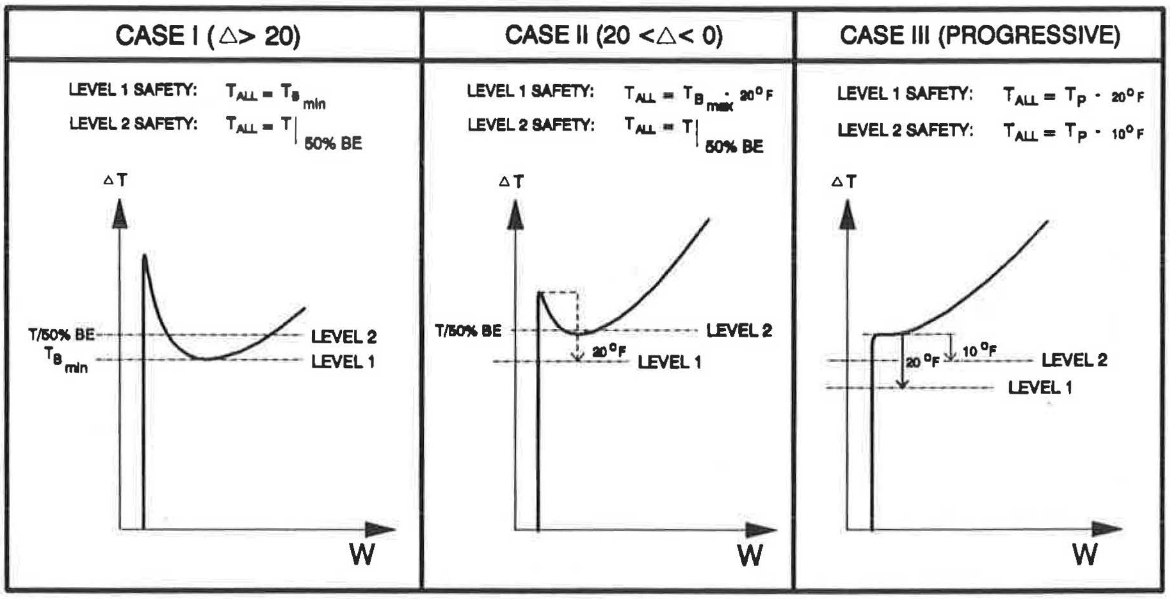


FIGURE 12 Safety criteria illustrations of levels of buckling safety.

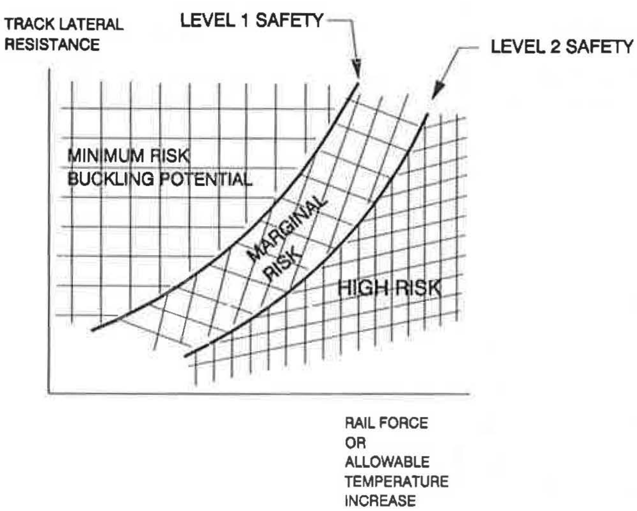


FIGURE 13 Illustration of prototype buckling safety criteria.

- Level 2 safety (moderate-risk buckling potential) is based on T (50 percent buckling energy) and T_p-10°F for allowable temperature increase values for Cases I, II, and III, respectively. The T limit is based on the supposition that Case I and II tracks can probably tolerate temperature increases above the $T_{B,\min}$ value, as seen in some tests presented in the next section. The temperature corresponding to the 50 percent buckling energy value (recall that at $T_{B,\min}$ the buckling energy is 100 percent) is an interim recommendation pending further research. The T_p-10°F value for Case III tracks is based partly on test results for progressive buckling response, and on industry consensus that even at the T_p value, Case III tracks (typically with high degrees of curvatures and low operating speeds) can probably tolerate train traffic at an acceptable level of risk.

The Level 2 safety limit values proposed are recommended only for those railroad institutions willing to maintain tracks to closer tolerances and implement CWR installation practices that adequately control the rail neutral temperature and hence the maximum force levels. Figure 11 provides specific examples of Levels 1 and 2 safety limits for Cases I, II, and III category tracks. Figure 13 illustrates sample prototype safety criteria in terms of allowable temperature increase (or rail force) for various levels of track resistance.

DYNAMIC BUCKLING TESTS

Dynamic buckling tests were carried out during 1983–1984 and 1986–1987 in the United States at the Transportation Test Center, in Pueblo, Colorado, on tangent and curved CWR tracks. Detailed summaries of these tests are given elsewhere (*12–14*).

The principal objectives of these tests were

- Experimental validation of dynamic buckling theory and identification of significant parameters that influence CWR track buckling response under thermal and vehicle-induced loads.
- Determination of required margin of safety for verification of proposed safety concepts and limits.

Test Methodology

The test methodology consisted of heating the rail by electric current using substations or diesel locomotives. The test track lengths varied but were of the order of 1,000 ft to minimize end effects and obtain uniform rail force distribution in the central segment of the test zone. Lateral misalignments were set intentionally in the test track, and all other existing misalignments were mapped using a track geometry car or stringlining techniques. The tracks were destressed and instrumented with longitudinal rail force and vehicle wheel load gages as well as displacement transducers to measure longitudinal, lateral, and vertical movements of the rails. Thermocouples were used to measure rail temperature. Data loggers and strip chart recorders were employed to record data at frequent intervals. Track resistance was measured by both panel pull tests and STPTs. The number of cars in the test consist varied up to 70, depending on the tests.

Dynamic Buckling Theory Verification Tests

Comparison of Buckling Strength Under Hopper and Locomotive

To compare the relative influence of the central bending wave under a loaded 100-ton hopper and locomotive, equal levels of misalignment were set under each of the vehicles. Vertical and lateral displacements were measured as the rails were heated. Figure 14 shows a comparison of lateral displacements under each vehicle as a function of temperature. The misalignment growth under the hopper car is much more severe, indicating the influence of the longer uplift wave present under the 100-ton hopper car. The uplift wave is a contributing factor in the misalignment growth mechanism and hence a critical component of the dynamic buckling analysis. Subsequent dynamic tests and Figure 15 further confirm this uplift wave influence.

In another test, the measured response of the track with a large misalignment under a stationary hopper car favorably compared with the theoretical prediction (Figure 16). This test facilitated determination of lower buckling temperature and progressive buckling characteristics.

Comparison of Static and Dynamic Strengths of CWR

A weak 5-degree curved track was tested dynamically by a locomotive and hopper car at slow speeds. After an increase in temperature of up to 40°F above neutral and five train passes, initial misalignment did not increase. Train passes made at temperatures above 40°F increased the misalignment; at 62°F, the curve buckled to a deflection of 9 in., as shown in Figure 15. The buckling response was in agreement with the dynamic theory, but more important, these tests gave the first indication of a 10 to 20°F dynamic factor of safety requirement (i.e., at the buckling temperature of 62°F minus 10 to 20°F, track deflections were still very small).

Effect of Uplift Wave and L/V

In several tests the growth of imperfections under the passage of different cars was monitored using strip chart recorders. Figure 17 shows a typical result from the charts. The significant influence of the central bending wave of the hopper car can be seen. In contrast, the locomotive did not increase the deflection, which is in agreement with the theoretical predictions.

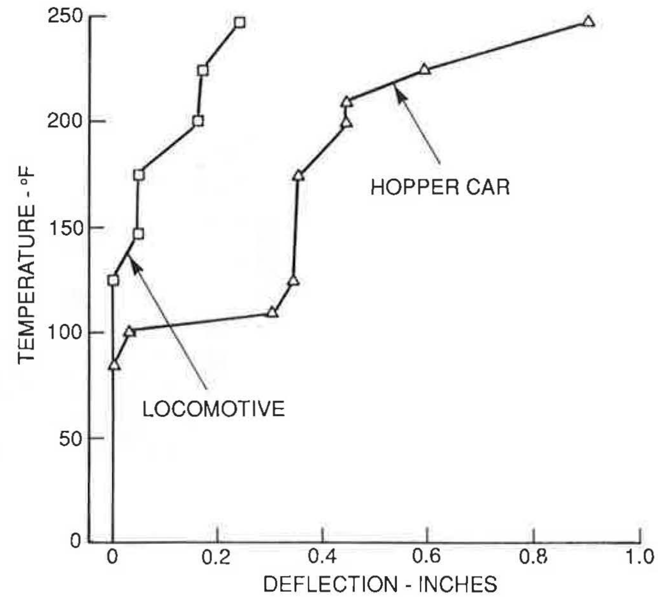


FIGURE 14 Response of track under vehicles.

Safety Concept Validation

Safety concepts and limits were partially verified on tangent, 5-, and 7.5-degree curved track as follows:

Tangent Track Tests (Tangents I and II)

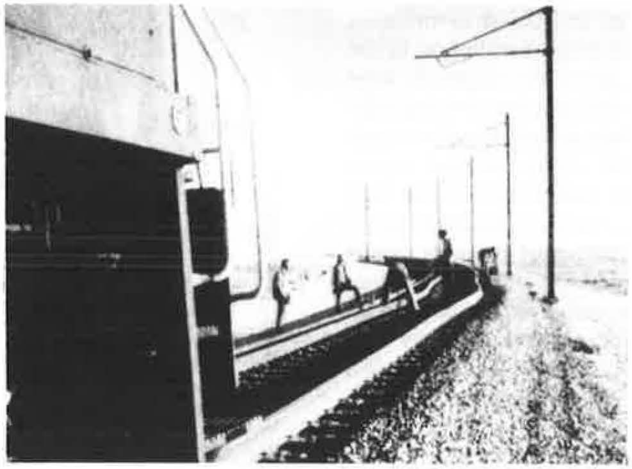
In Tangent I with a lateral resistance (peak) value of 69.1 lb/in. and in Tangent II with a peak value of 89 lb/in., train passes were made at incremental heating levels. Results are shown in Table 1. The conditions represent Case I type tracks as referred to previously. No significant movement occurred at Level 1 safety limits. At higher temperatures attained in the test, the increase in misalignment was small; however, the vehicles were not operated at maximum allowable speeds.

5-Degree Curve Tests (Curves I and II)

Results for Curves I and II representing different peak resistance values are shown in Table 1. Again, the results are satisfactory from the Level 1 safety viewpoint. This is seen from the maximum temperatures reached in the test, which were in excess of the Level 1 temperatures.

7.5-Degree Curve

The objective in this test was not only to validate the Level 1 safety limit, but also to determine the ultimate buckling strength under a moving consist. The Level 1 safety limit of



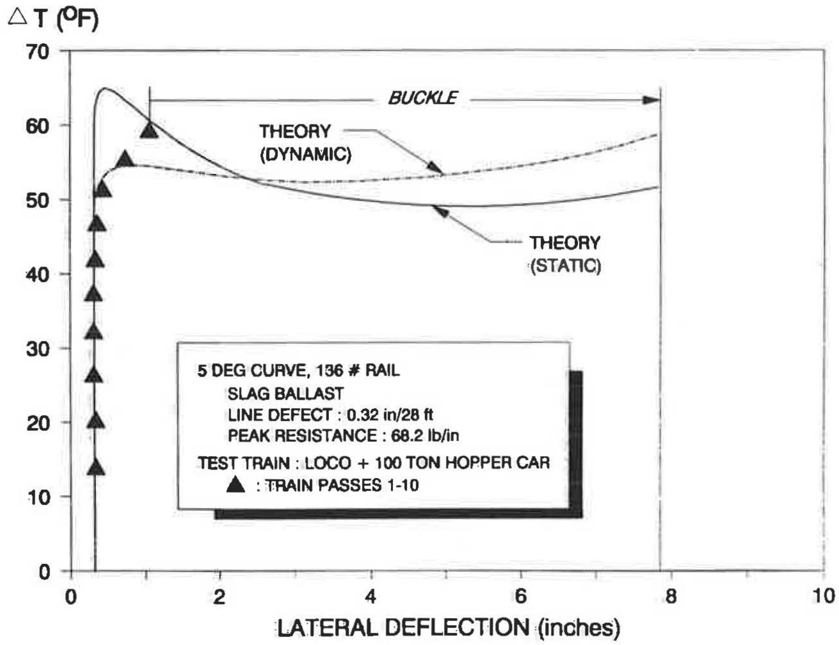
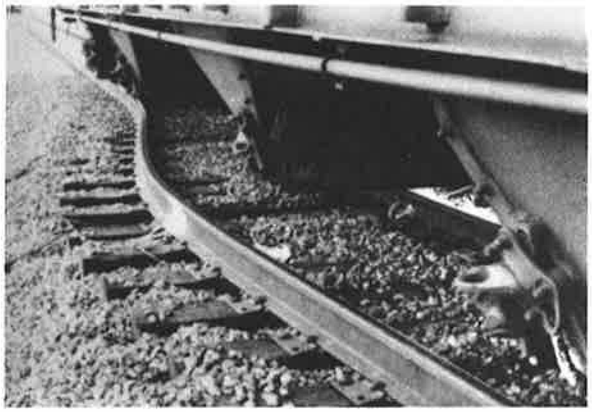


FIGURE 15 Dynamic buckling of curved track.

52°F was reached without causing significant increased misalignment due to vehicle passage. Analytical and experimental results are shown in Figure 18. At 62°F above the stress-free temperature, cumulative increased misalignment was experienced under the passage of each car. This misalignment resulted in a total deflection of 4.5 in. under the 12th car in the final run of the 24-car consist, before derailment at another location in the test zone stopped the test. Figures 19 and 20 present a view of the track and a derailed car. The test shows that the track can withstand Level 1 safety limit temperatures, and that buckling occurred below $T_{B,\max}$ and above $T_{B,\min}$.

CONCLUSIONS

- A versatile buckling model that can be run on a PC has been developed. The new model overcomes the deficiencies in other models, namely, absence of vehicle load effects, inadequate idealization of nonlinear lateral resistance, and lack of rational safety criteria. The model accounts for the loss of lateral resistance caused by a track uplift bending wave under vehicle loads. It also considers the softening behavior of the lateral resistance at large displacements, a phenomenon that has not been recognized in previous work. The model computes the energy required to precipitate buckling and thus



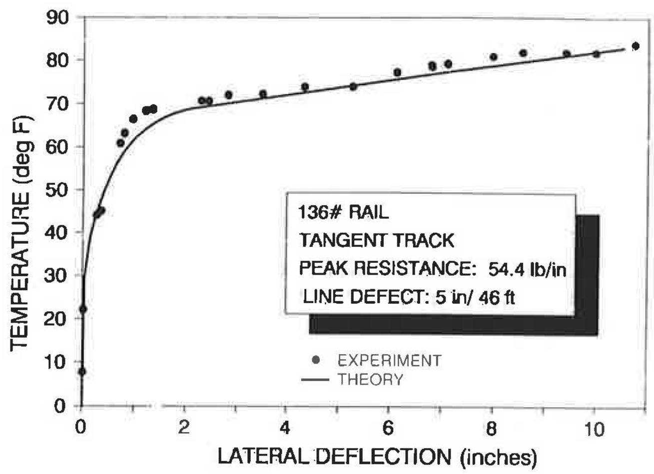


FIGURE 16 Dynamic buckling test (tangent).

evaluates the degree of safety of CWR at a given rail temperature.

- The model has been validated through several controlled, full-scale, dynamic buckling tests in which rails were artificially heated, and a long consist of cars made several passes at full speed over tracks with initial misalignments. Tangent, 5-, and 7.5-degree curves were tested in the validation of the dynamic model. Static tests, which showed higher buckling strengths in the absence of train traffic, were also performed.
- Vehicle vertical loads create precession or recession and central uplift bending waves in the track. For cars with large truck center spacing (hopper) the central uplift wave is critical, whereas for smaller truck center spacing cars (locomotive) the precession wave has more significant influence on buckling.
- In general, the growth of lateral misalignment under a vehicle is caused by a central bending wave rather than L/V. The influence of L/V can be significant for high impact loads and weak resistance tracks.
- The softening behavior of the lateral resistance is important in the analysis because it will have a significant influence on the lower buckling temperature. Idealizing the resistance as a constant at the peak value overestimates this temperature.
- The upper buckling temperature is sensitive to the peak value of the lateral resistance, the track misalignments, and the car parameters.
- Buckling safety limits are best approached on the basis of the energy levels required to buckle the track. Level 1 and 2 safety limits are introduced in this paper for low and moderate risks associated with track safety. The Level 1 limit has a margin of safety of at least 20°F, whereas Level 2 has a lower margin of safety. Level 1 safety limits have been verified for the tangent, 5-, and 7.5-degree curves through full-scale tests.

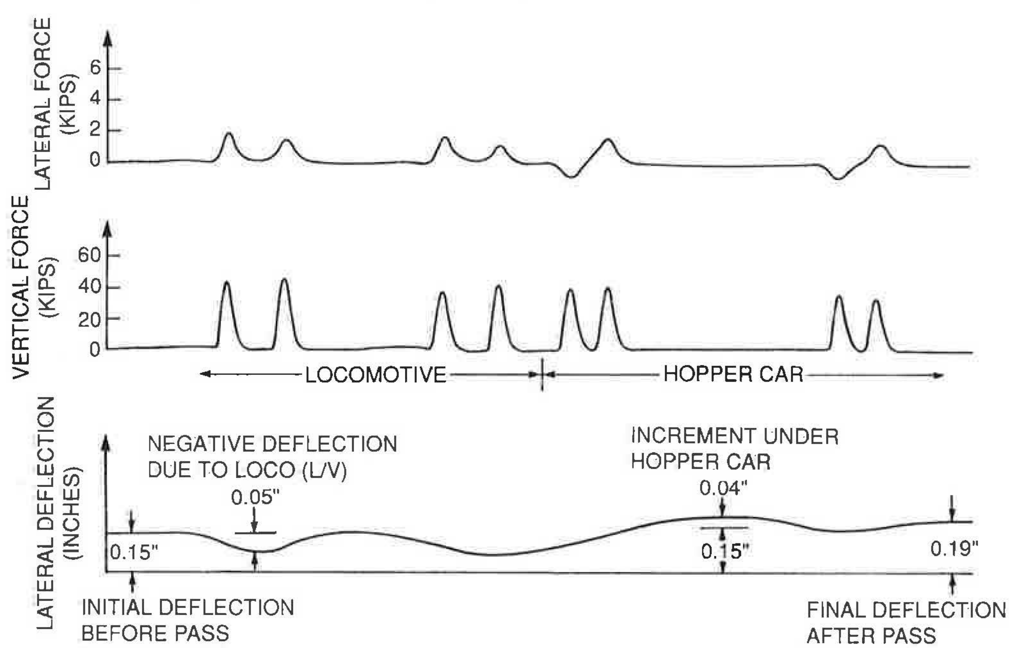


FIGURE 17 Strip chart record for pass no. 8 (curve with finite margin of safety).

TABLE 1 SUMMARY OF SAFETY LIMIT TESTS

TANGENT I	*PASS #*	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*
N = 48	**ΔT (°F)**	61	71	81	80	88	86	93	92
V = 20	**P (kips)**	157	182	208	207	228	222	239	237
	δ_0 (in.)	0.88	0.89	0.91	0.91	0.94	0.95	0.98	0.99

TANGENT II	*PASS #*	*1*	*2*	*3*	*4*	*5*	*6*	*7*	*8*
N = 67	**ΔT (°F)**	70	76	83	82	85	95	100	100
V = 55	**P (kips)**	181	196	213	211	221	246	259	259
	δ_0 (in.)	0.81	0.83	0.86	0.87	0.78	0.79	0.81	0.82

CURVE I	*PASS #*	*1*	*3*	*5*	*7*	*9*	*11*	*13*	*15*	*16*	*17*
N = 63	**ΔT (°F)**	10	18.5	31	40	50	61	61	68	69	70
V = 20	**P (kips)**	25	48	81	104	129	158	157	175	179	180
	δ_0 (in.)	0.55	0.48	0.47	0.48	0.48	0.53	0.49	0.54	0.52	0.54

CURVE II	*PASS #*	*1*	*2*	*3*	*4*	*5*	*6*	*7*
N = 52	**ΔT (°F)**	39	64	68	56	66	72	80
V = 20	**P (kips)**	101	165	176	148	170	186	205
	δ_0 (in.)	0.50	0.55	0.58	0.70	0.75	0.79	0.84

TEST TRACK	*Fp (lb/in)*	*LEVEL 1 SAFETY LIMIT*	*ΔT_{test}*
TANGENT I	69.1	63	93
TANGENT II	80.0	65	100
CURVE I	83.7	59	70
CURVE II	100.0	60	80

N = Number of cars; V = Speed in mph; δ_0 = Line defect amplitude; P = Rail force

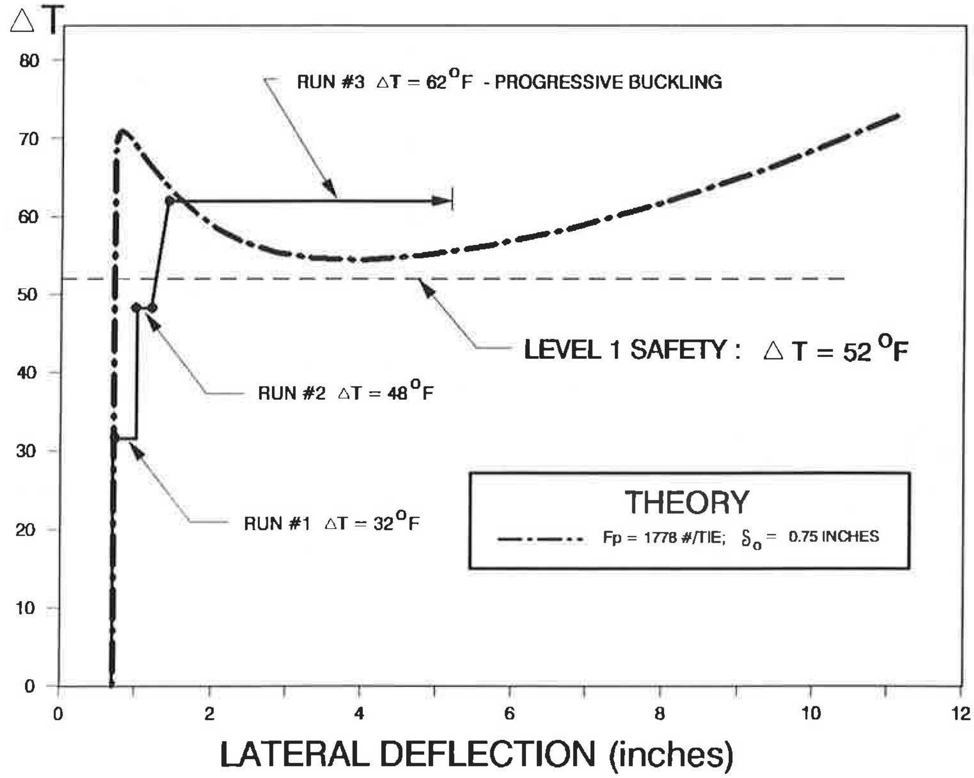


FIGURE 18 Dynamic buckling test analysis versus experiment.

FIGURE 19 Track condition after derailment.

FIGURE 20 Last car derailed.

ACKNOWLEDGMENTS

The authors wish to thank D. Jeong and M. Thurston for the development of the dynamic buckling software on the PC.

REFERENCES

1. A. D. Kerr. *Analysis of Thermal Track Buckling in the Lateral Plane.* Report DOT/FRA/ORD-76/285. FRA, U.S. Department of Transportation, 1976.
2. G. Samavedam. *Buckling and Post Buckling Analyses of CWR in the Lateral Plane.* Technical Note TN-TS-34. British Railways Board, London, 1979.
3. A. Kish, G. Samavedam, and D. Jeong. *Analyses of Thermal Buckling Tests on U.S. Railroads.* Report DOT/FRA/ORD-82/45. FRA, U.S. Department of Transportation, 1982.
4. G. Samavedam, A. Kish, and D. Jeong. *Parametric Studies on Lateral Stability of Welded Rail Track.* Report DOT/FRA/ORD-83/07. FRA, U.S. Department of Transportation, 1983.
5. A. Kish, G. Samavedam, and D. Jeong. *Influence of Vehicle Induced Loads on the Lateral Stability of CWR Track.* Report FRA/ORD-85-3. FRA, U.S. Department of Transportation, 1985.
6. F. Amans and R. Sauvage. Railway Track Stability in Relation to Transverse Stresses Exerted by Rolling Stock: A Theoretical Study of Track Behavior (in French), *Annales des Ponts et Chaussées*, No. 1, Jan.–Feb. 1969.
7. A. M. Zarembski and G. Magee. *An Investigation of Railroad Maintenance Practices to Prevent Track Buckling.* AAR Report R-454. Association of American Railroads, Washington, D.C., 1980.
8. J. Nagy. Experimental Investigations on CWR Track Behavior Due to Thermally Induced Loads, V (in Hungarian), In *Yearbook of the Hungarian Railway Scientific Research Institute*, 1970.
9. J. Nagy. Experimental Investigations on CWR Track Behavior Due to Thermally Induced Loads, VI (in Hungarian), In *Yearbook of the Hungarian Railway Scientific Research Institute*, 1974.
10. J. Eisenmann. The Significance of the Rail Lifting Wave. *Rail International*, No. 10, Oct. 1976, pp. 576–581.
11. G. Samavedam, A. Kish, M. Thurston, and D. Jeong. Recent Advances in Track Buckling Mechanics. Applied Mechanics Rail Transportation Symposium, AMD, Vol. 96, RTD, Vol. 2, 1988.
12. G. Samavedam, A. Kish, and D. Jeong. *Experimental Investigations of Dynamic Buckling of CWR Tracks.* Report DOT/FRA/ORD-86/07. FRA, U.S. Department of Transportation, 1986.
13. A. Kish and G. Samavedam. *Analyses of Phase III Dynamic Buckling Tests.* Final Report DOT/FRA/ORD-89/08. FRA, U.S. Department of Transportation, 1989.
14. G. Samavedam and A. Kish. *Dynamic Buckling Test Analyses of a High Degree CWR Track.* Report DOT/FRA/ORD-85/03. FRA, U.S. Department of Transportation, 1989.

Continuous Welded Rail Track Buckling Safety Assurance Through Field Measurements of Track Resistance and Rail Force

G. SAMAVEDAM AND A. KISH

Techniques and hardware for field measurement of two important continuous welded rail (CWR) track parameters for safety from buckling—the track lateral resistance and the rail neutral temperature—are presented. It is shown here that by controlling the two parameters above their respective permissible minimum values, CWR track buckling safety can be ensured. For the measurement of lateral resistance, a lightweight, portable device that tests the ties individually has been developed. Field data collected using the single-tie push test revealed that the scatter is within permissible limits if the data for three randomly selected ties in a 50-ft section of CWR are averaged. The data have also shown that the ties exhibit a softening resistance characteristic, a feature that has been ignored or not detected in work by other researchers. Rail neutral temperature can be measured using the principle that the vertical deflection of a rail beam freed from ties is measurably sensitive to the longitudinal rail force when a vertical load is applied to the rail section. On the basis of that principle a rail uplift device (RUD) that gives the absolute rail force without site-specific calibration has been developed. The rail force test data from RUD are in agreement with the theoretical predictions.

The Transportation Systems Center (TSC) provides technical support to FRA in the development of performance-based safety guidelines and specifications for continuous welded rail (CWR) track. A major problem with CWR track is lateral buckling under high thermal and vehicle loads. TSC recently completed a major analytical, experimental, and safety assessment study, which is discussed by the authors in another paper in this Record. The limitations of existing theories are discussed, and an advanced model that runs on a personal computer (TSC dynamic buckling model) and accounts for vehicle loads, nonlinearity in the lateral resistance, and all other significant parameters is described. Results from controlled full-scale buckling tests (*1–3*) that used artificial heating and moving train consists are also reported. The tests validated the theoretical model (*4*) and safety concepts and limits.

On the basis of the TSC computer model, and on the knowledge of two parameters (track lateral resistance and rail neutral temperature), it is now possible to assess the in situ buckling strength of CWR track for an improved assurance of safety from buckling.

G. Samavedam, Foster-Miller, Inc., 350 Second Avenue, Waltham, Mass. 02254. A. Kish, Transportation Systems Center, DTS-76, 55 Broadway, Cambridge, Mass. 02142.

Recent developments in the concepts, methodology, and hardware for the measurement of track resistance and rail longitudinal force (neutral temperature), and their applicability to CWR track buckling safety assurance are described in this paper.

BUCKLING SAFETY ASSESSMENT

The TSC dynamic buckling model can predict upper and lower buckling temperatures for given input data. The data can be divided into (a) primary inputs: rail size, car parameters (truck center spacing and wheel loads), track curvature, misalignments, and lateral resistance, and (b) secondary inputs: longitudinal resistance, track modulus, and tie-ballast friction coefficient. The primary input data have significant influence on track buckling response and therefore must be accurately known.

A lightweight, portable device has been developed that can be used to determine the lateral resistance of the track, the most difficult of the five primary inputs for the track engineer to estimate. This hardware and the associated test methodology can be used to determine the complete nonlinear resistance. Tie motion of only a fraction of an inch allows determination of the peak value. To determine the complete nonlinear response, ties may be displaced to larger deflections, or the response may be determined empirically, on the basis of correlation with existing field test data. The importance of the complete nonlinear resistance for buckling predictions is discussed by the authors in another paper in this Record.

Once the lateral resistance in the field and the critical buckling temperatures from the TSC model have been determined, the safety criterion to be applied is as follows.

For safe operations of CWR tracks with regard to buckling, the allowable temperature increase (ΔT_a) should be greater than the difference between the maximum rail temperature (T_M) and the neutral or the force-free temperature (T_N).

$$\Delta T_a > T_M - T_N \tag{1}$$

T_M depends on the ambient conditions for which data are generally available. T_N is not necessarily the installation temperature. The neutral temperature can change substantially from its original value at installation because of several mechanisms discussed later. Its value must be upgraded each time track operations such as destressing, reanchoring, and lining are performed. Hardware and procedures, described later, have also been developed to determine the rail neutral temperature in the field. This method gives the absolute rail force and does not involve any specific site-dependent calibration and rail cutting.

Thus, Equation 1 can be used for buckling safety assessment once values for ΔT_a and T_N are known. Among the primary parameters governing ΔT_a, for the most commonly used wood-tie track with cut spike construction, the ballast lateral resistance is the only variable that generally can be controlled by the track engineer. Hence, in revenue service conditions, ΔT_a is essentially controlled by the lateral resistance. A minimum value for ΔT_a can therefore be ensured by stipulating a minimum permissible value for the resistance. Likewise, if a minimum value for T_N is also stipulated, Equation 1 will be satisfied for all values of resistance and the neutral temperature above the respective permissible values. The track can be rapidly tested for the two permissible values through the use of available equipment. A go or no-go criterion can be used for buckling safety. If the lateral resistance is below the critical value, ballast can be added, or the existing ballast can be consolidated by traffic or other means. Likewise, if the neutral temperature falls below the critical value stipulated, rail destressing can be performed. A slow order should be imposed on trains until the track attains the minimum stipulated values. The minimum required lateral resistance and neutral temperature values can be made available to the track supervisor in the form of simple charts or graphs.

LATERAL RESISTANCE

Track lateral resistance has been measured by a number of researchers in the United States and abroad. The currently recommended measurement scheme mobilizes a single tie; some previous techniques require lateral movement of a cut panel or the entire track section by a concentrated lateral load. In the case in which a single tie is mobilized, the resistance is directly represented by the load-deflection response of the tie, whereas in the case of the panel, the load-deflection response is a combined effect of rail flexural rigidity, rail longitudinal force, and nonuniform resistance offered by several ties. The panel deflection response is not directly usable as an input parameter in the buckling analysis, which requires individual tie resistance data. In past buckling investigations, single-tie push tests (STPTs) were not favored for the lateral resistance measurement because of the scatter, or variations, in the individual tie resistance values.

The advantages of the STPT over the panel test are

- STPTs yield a more fundamental characteristic of the ballast resistance;
- The test is easy to set up and perform;
- The hardware is portable and can be used by track crew with minimal training;
- If a discrete panel is used, rail cutting is destructive; and
- For the continuous panel, the data are substantially skewed by rail longitudinal forces that influence the deflection response.

The disadvantage of the STPT is the variation of results from tie to tie. However, an arithmetic average of the individual test results is adequate to determine the buckling and safe allowable temperatures from the safety limit charts currently under consideration. It will be shown in this paper that for a 50-ft section of CWR track, three randomly selected ties are adequate to yield a resistance value that can predict the lower buckling temperatures within 10°F.

Test Hardware

Although STPTs were performed many years ago in the United States and abroad, they were restricted to very small tie displacements and did not cover the "softening" portion of the resistance characteristic. Further, the equipment used was bulky and not suitable for generation of a large data base. A new, lightweight, portable device with an X-Y plotter was therefore developed. The STPT device, shown in Figure 1, consists of a hydraulic control unit with a pump and a rig with a hydraulic cylinder. Once the spikes, rail anchors, and tie plates are removed, the rig assembly grabs the test tie, which is now free to move laterally under the rails. The hydraulic piston mounted on the rig creates the force required to move the tie against one of the rails. Hydraulic pressure can be provided by the hand pump or by an electric pump to speed the operation. Most reported testing was performed by the latter method.

A pressure transducer or load cell in line with the piston and a pressure gauge in the control unit (as a backup) indicate the load applied; a rotary potentiometer mounted on the tie measures the displacement with respect to the stationary second rail. The load-displacement relationship is plotted using the X-Y plotter.

Typical Results

TSC conducted a large number of track characterization tests using the STPT device at the Transportation Test Center (TTC) in Pueblo, Colorado, and on a number of railroads. Detailed load deflection response curves for individual ties under a range of ballast and test conditions are presented by Pietrak et al. (*6*), and data analysis results and correlations among the parameters controlling the lateral resistance are presented elsewhere by Samavedam and Kish (*7*).

Typical results for relatively strong, medium, and weak tracks are shown in Figure 2. There are two salient points on the characteristics: the peak (F_P), occurring at displacements on the order of 0.25 in., and the limiting value (F_L), at about .5 in. or less. The softening behavior becomes pronounced for high F_P (>1,000 lb), whereas for low F_P (<1,000 lb), the resistance is practically constant with $F_L \cong F_P$.

Typical STPT data from tests conducted at TTC are shown in Figure 3. These data are averaged for a large number of tests in the test zones, each of which is several hundred feet long.

The data show the resistance values up to 2-in. tie displacement for granite and slag at fractional and large consolidation levels. On the basis of such data, the influence of consolidation, type of ballast, and minimum number of STPTs required to characterize the track resistance will be presented in the following sections.

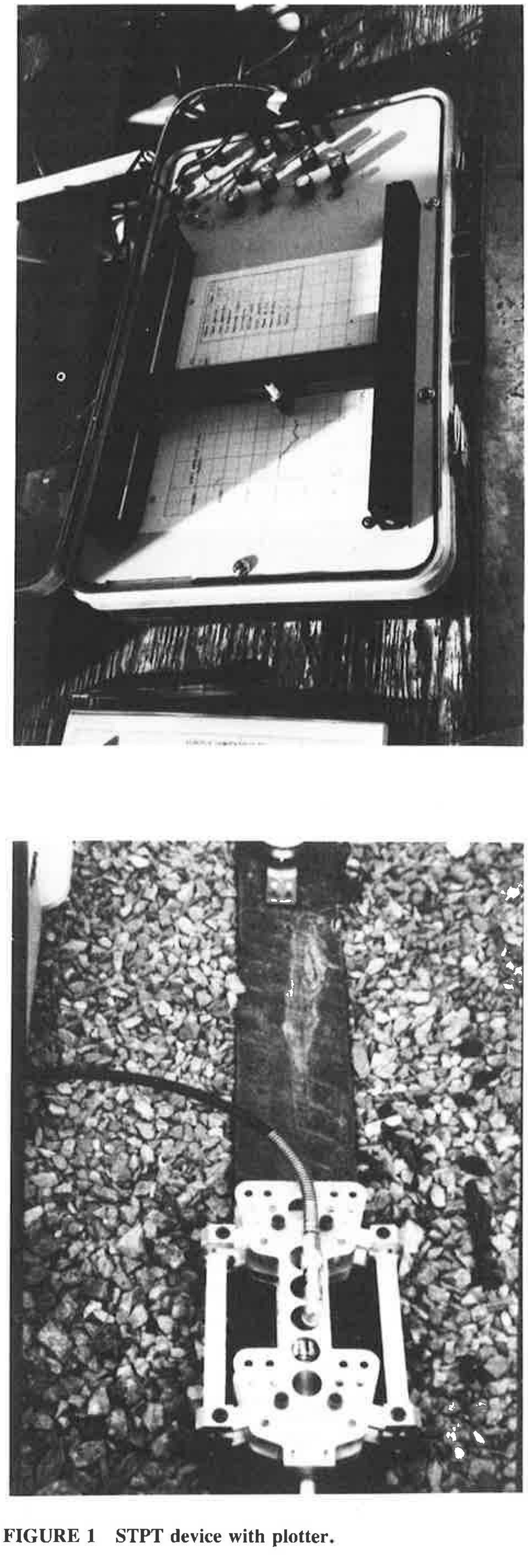


FIGURE 1 STPT device with plotter.

Correlation Study

From the previous data, it is seen that ties need to be laterally displaced over a large distance ($\approx$5 in.) to capture the limiting resistance values. This may be undesirable in revenue service track. Therefore, a correlation between the peak value (F_P), which can be easily determined at small displacements, and the limiting value (F_L) will be developed here for use in the buckling model. Attempts will also be made to correlate the peak value to the traffic tonnage [in million gross tons (MGT)], but there are some difficulties, as seen later. Finally, the scatter in the peak values for a given track will be presented, and the sampling size, that is, the number of required STPTs over a given track segment for the purpose of averaging the peaks, will be determined.

Limiting Versus Peak Resistance Values

Considerable test data have been generated to correlate the limiting resistance (F_L) with the peak value (F_P). This correlation depends on the type of ballast material. For granite ballast, the linear regression analysis of the data has given the following equation:

$$F_L = (0.3\ F_P + 500)\ \text{lb} \quad \text{for } F_P > 726\ \text{lb} \qquad (2)$$

For $F_P \leq 726$ lb, the case of weak track, it can be assumed that $F_L = F_P$.

For slag ballast, the equation is

$$F_L = (0.06\ F_P + 600)\ \text{lb} \quad \text{for } F_P > 638\ \text{lb} \qquad (3)$$

For $F_P \leq 638$ lb, $F_L = F_P$.

The ability of granite to provide higher limiting lateral resistance is seen from the equations plotted in Figure 4. It must be noted that the foregoing empirical equations are based on the tests on slag- and granite-ballasted tracks at TTC, which had a shoulder width of about 12 to 14 in. The equations may not be strictly applicable to other track conditions. A significant scatter also exists in the test data. The equations are provided to show that it may not be necessary to push test ties over large lateral displacements to determine the full characteristic. Knowledge of the peak value alone may be adequate and can be easily determined at small displacements without significantly damaging the track.

Effect of Track Consolidation

It is known that consolidation under traffic (measured by tonnage accumulation in MGT) increases lateral resistance to some limit. Beyond this limit, consolidation has little effect. However, there is a problem in correlating MGT with the absolute value of track lateral resistance. The problem is that immediately after tamping or other maintenance operation, the track resistance drops to a low but unknown value. The subsequent increase in the resistance from this condition would depend on the MGT level of consolidation. Due to the nonlinear relationship between the lateral resistance and MGT, it is difficult to predict the absolute resistance at a given MGT.

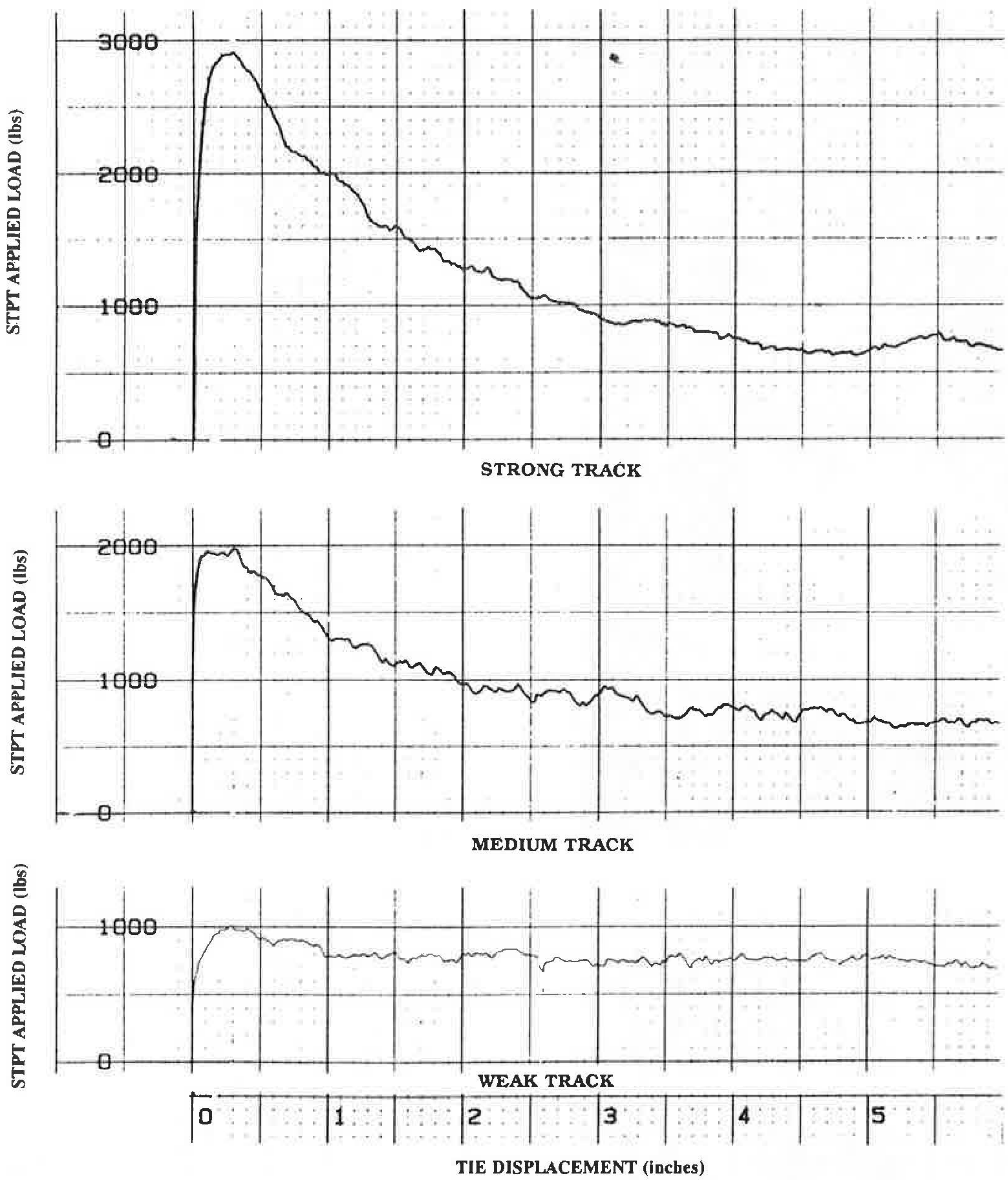


FIGURE 2 Typical STPT track response.

Tests to understand the influence of consolidation on the peak resistance values were conducted on three zones of slag, traprock, and granite ballast, respectively, that were subjected to the same traffic levels. The averages of STPT results are shown in Figure 5. Clearly, the resistances at zero MGT for the three zones were not equal, even though the same tamping procedure was employed at each zone. The starting values (1,800 lb for slag, 1,520 lb for granite, and 1,200 lb for traprock) should be considered as site-specific and cannot be attributed to a particular ballast. Previous track operations at these locations, tie condition and age, and resistance levels before tamping can play an important role in the reduced resistance levels after tamping.

Data on peak resistance values collected at various increments in MGT are shown in Figures 5 and 6. These data clearly indicate that the resistance values increase monotonically up to some level. Figure 6 is of particular interest because it shows the significant gain in peak lateral resistance for small increments in consolidation. Such data will be helpful in determining slow-order duration for reduced train speeds soon after tamping or similar track operations.

Sampling Size

Because of inherent variations in the ballast and tie conditions, not all the STPTs in a given section will yield the same values. The longer the section is, the greater will be the scatter in the individual resistance values. Besides the section length, the scatter will depend on the track maintenance standards

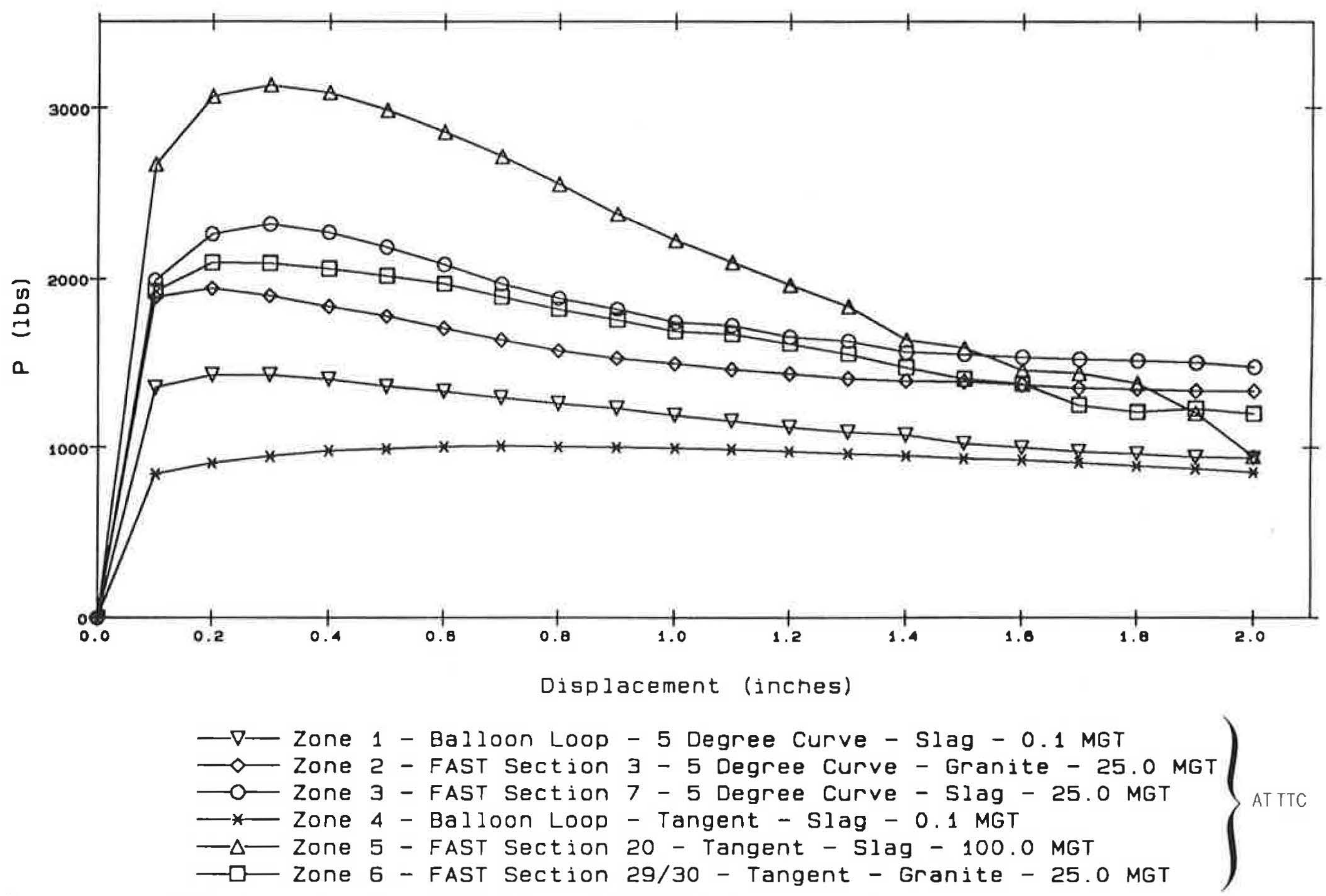


FIGURE 3 Ballast resistance characterization tests (average STPT behavior summary for 2-in. tie displacement).

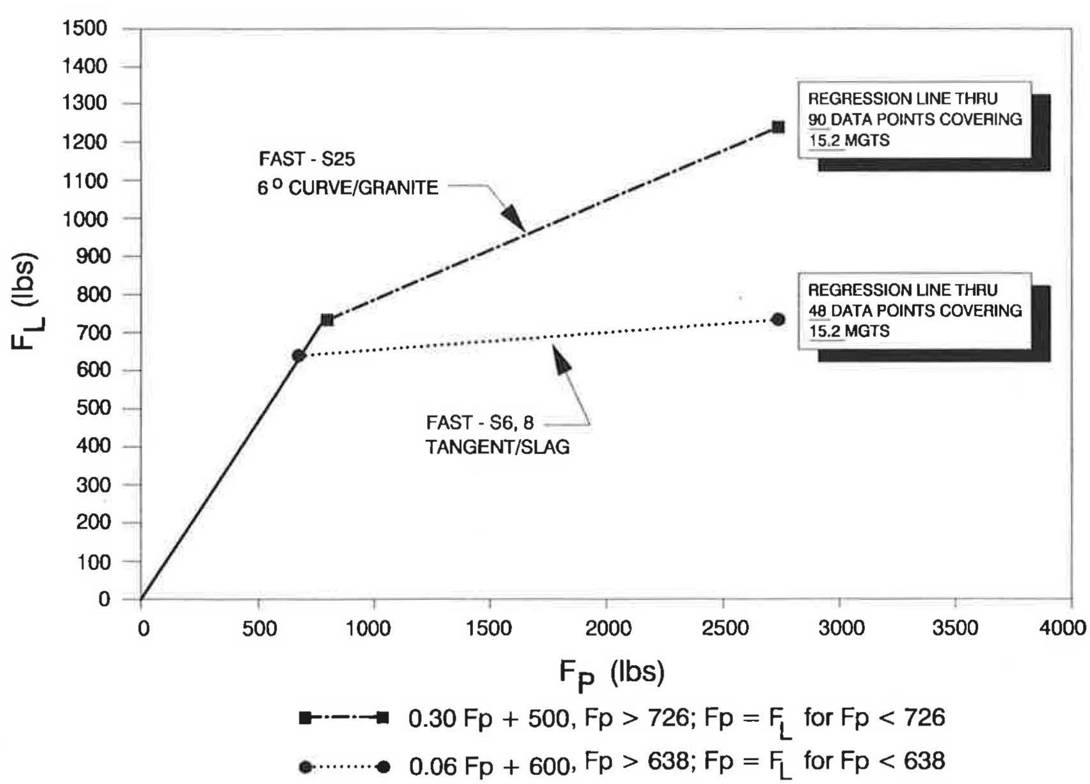


FIGURE 4 Peak versus limit resistance (limit values occurred at tie displacements of 4 in. or larger).

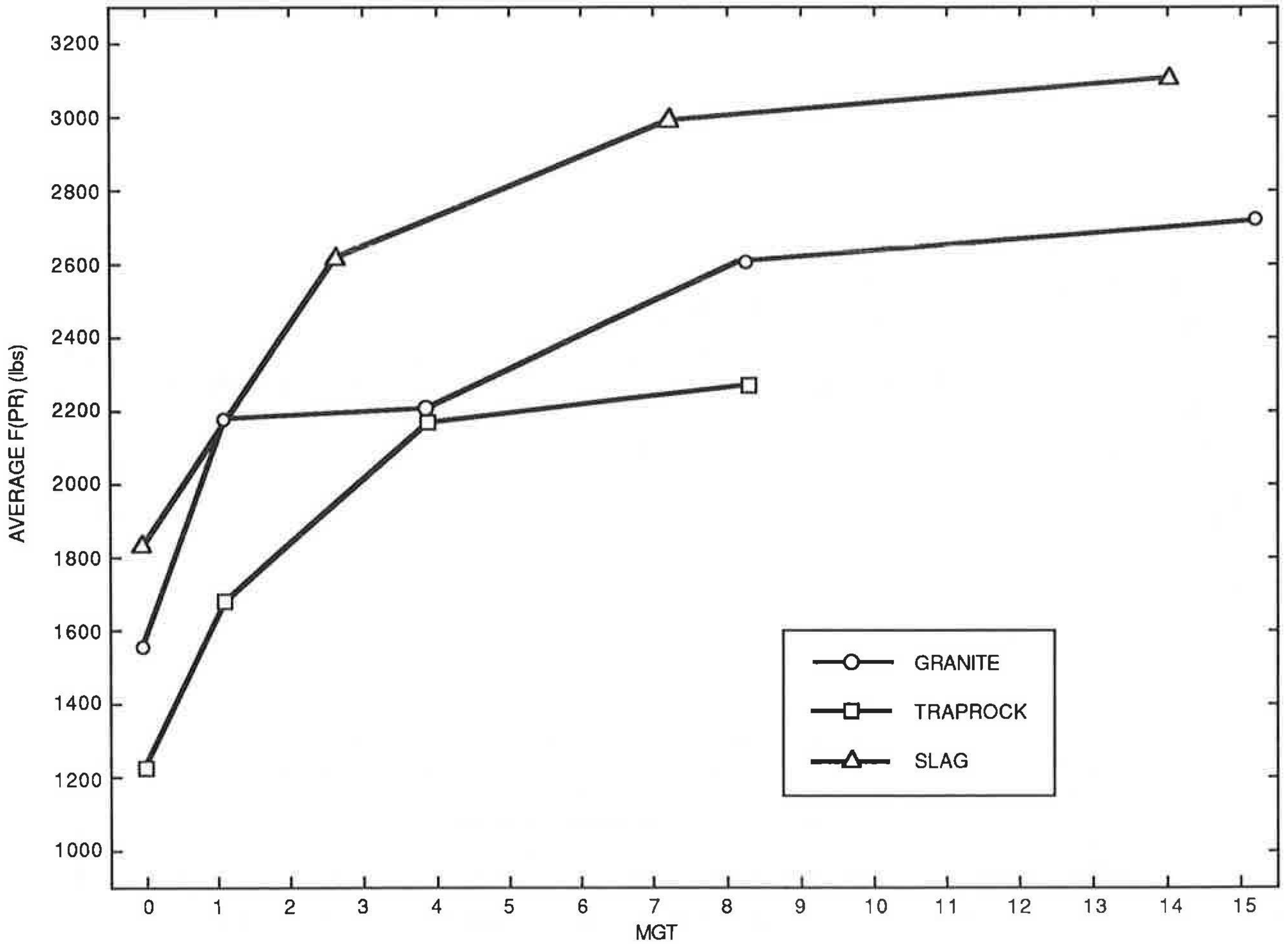


FIGURE 5 Ballast consolidation influence.

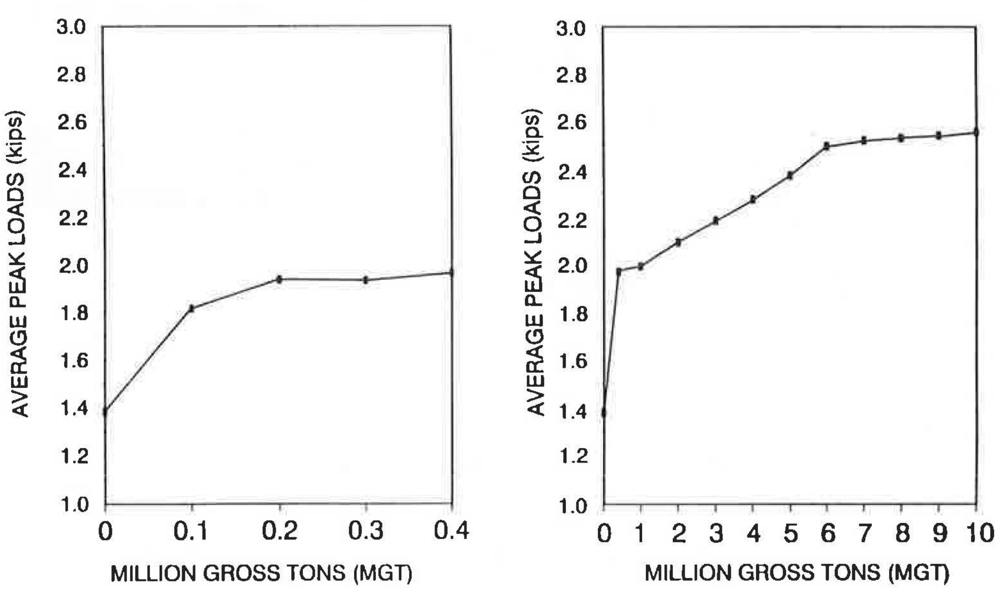


FIGURE 6 Fast ballast resistance characterization tests—consolidation influence (tangent track, granite ballast).

of the railroad. Tests have also indicated that for a given track section, the scatter increases with the increasing consolidation level.

Although the computer model described by the authors in another paper in this Record can account for the individual tie variations, it is not practical or desirable to test a large number of ties for buckling safety predictions. The question therefore arises whether a minimum (optimum) number of single-tie tests can be established for a given section length, the average of which can be considered as the resistance for the section under consideration. Such an average can then be used as an input parameter in the buckling model.

To address the foregoing question, a large number of tests was performed at TTC on different track sections and at different consolidation levels. Test sections about 50 ft long were considered for the case studies. In each section, alternating ties were tested, and the average of the 15 tested ties was considered to be the lateral resistance for the section.

If fewer than 15 ties in each section were tested, the average of these results would clearly differ from the overall average (F_o). Suppose three ties whose peak resistance values are F_1, F_2, and F_3 were selected randomly. The percentage error with respect to the overall average is equal to $(F_m - F_o)/F_o$, where the average of F_1, F_2, and F_3 is F_m.

The percentage error was determined in five trials through the use of a random number generator (each trial yields one set of F_1, F_2, and F_3), and the maximum error produced in these trials for each of the six test sections is plotted in Figure 7. This is repeated for all the zones previously referred to in Figure 3.

From Figure 7 it is seen that the maximum error is about 20 percent. This error generally translates into an error of about 10°F in the lower buckling temperature from the buckling model discussed by the authors in another paper in this Record. Factors of safety built into the safe allowable temperatures may make the 10°F uncertainty tolerable. Hence,

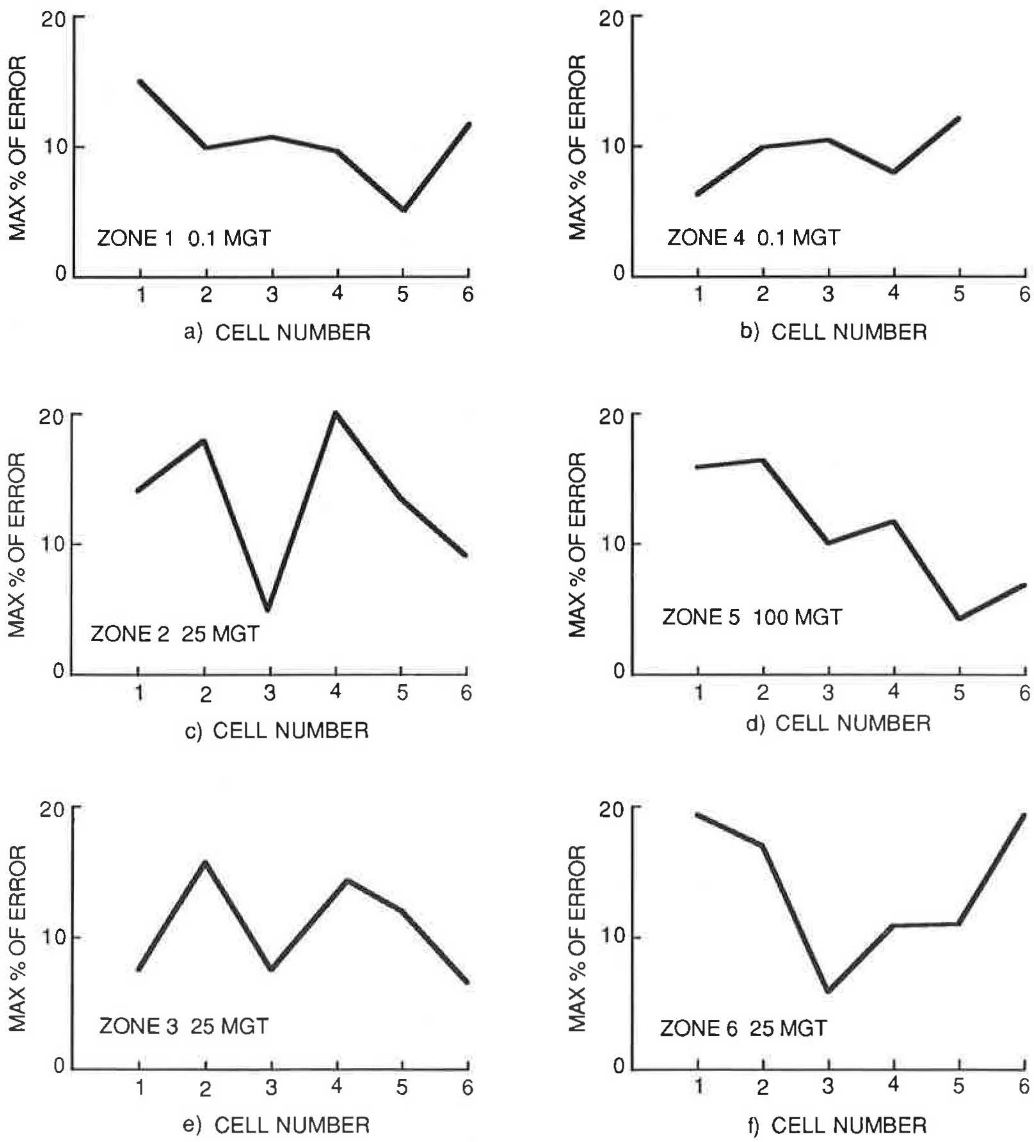


FIGURE 7 Error due to finite sampling of test ties.

it is concluded that a sample of three randomly selected ties for every 50-ft section may be adequate in the field application of STPT. Clearly, a linear extrapolation of this result would imply testing six ties for 100-ft sections. However, by visual inspection and proper engineering judgment, the number of STPTs required per unit of length can further be reduced as the length of the section increases. These and other practical considerations will be dealt with in upcoming studies.

Results for a sample size of five ties per 50-ft section, not presented here, indicate a maximum error of 10 percent, which is more than adequate from a practical point of view.

Figure 7 also indicates that tracks with low consolidation levels have a lower percentage error than highly consolidated tracks. This is fortunate because STPT is more important for tracks with low consolidation levels.

RAIL FORCE AND NEUTRAL TEMPERATURE MEASUREMENT

As stated earlier, the neutral or force-free temperature of CWR can be different from the initial temperature at installation. If the rail force P is known at a given rail temperature T, then assuming the rails are fully constrained, the neutral temperature T_N can be calculated from the equation

$$P = AE\alpha\,(T - T_N) \tag{4}$$

where

A = rail cross-sectional area,
E = modulus, and
α = coefficient of thermal expansion.

TABLE 1 MAINTENANCE ACTIONS THAT INFLUENCE RAIL NEUTRAL TEMPERATURE VARIATIONS

Maintenance Activity	Problem
CWR installation at extreme temperatures	Hard to control a uniform laying temperature via rail heating, cooling, and destressing
Destressing	Difficulty in ensuring uniform rail temperatures during welding and anchoring
Replacing broken rail	Rail stress free temperature is usually not known, hence it is difficult to adjust to it
Lining, lifting and tamping	Rail longitudinal stress distribution altered

Of course, the rails are not fully constrained, but the equation can still be used to define a variable neutral temperature. Mechanisms contributing to neutral temperature variations are discussed elsewhere (*8*). The mechanisms include rail longitudinal movements, track lateral shift and radial breathing in curves, and track vertical settlement. Rail longitudinal movement is caused by train braking and acceleration forces or by differential thermal forces (sun and shade). Track lateral shift can be caused by truck excessive hunting, lateral forces generated due to curving, or negotiation of lateral misalignments. Rail force can cause radial breathing of curves in weak ballast conditions. Vertical differential settlement of rails can occur on new or recently surfaced track or in areas of weak subgrade conditions.

These natural mechanisms demand that CWR neutral temperature be determined from time to time. Track maintenance operations, given in Table 1, can also affect the neutral temperature. It is desirable to determine the rail neutral temperatures after the track undergoes any of these operations. This is particularly important in spring and summer to ensure permissible values for buckling safety. Field data collected by TSC (*8*) using the strain gage affixed to rail on a number of revenue service tracks and tracks at TTC showed that the neutral temperature could drop from a typical installation value of 90°F to 50°F, thus significantly increasing the buckling risk on a hot day.

Measurement of Rail Force

Rail force measurement by Berry gage, strain gage, and the British Rail vibrating wire are well known but are not practical for use in the field, as explained elsewhere (*9*). They cannot provide the absolute rail force and need an initial reference level, usually obtained by cutting the rail. The vibrating wire technique requires that a hole be cut in the rail web. A number of other techniques (*10*) have been tried, some of which are listed in Table 2. These techniques generally suffer from problems of reliability, sensitivity to the rail residual stresses, and site-specific calibration requirements. To address these problems, a new technique has been recently developed, and a prototype test fixture has been used to validate the technique through field tests. The technique is founded on a well-known principle of mechanics, and it provides the absolute force without site-specific calibration. It is not destructive but requires removal of spikes and anchors from the test section rail.

The technique, which is based on rail uplift induced bending response, was originally described elsewhere by Kish and Samavedam (*9*).

Rail Uplift Method

If the rail is freed from the ties over some length, restrained vertically at the ends of the freed portion, and subjected to a concentrated uplift load at the center, the resulting deflection depends on the magnitude of the rail longitudinal force. Clearly, longitudinal compressive load will increase the deflection of the beam-column, and tensile force will reduce it. For a given length of rail, the vertical force required to pro-

TABLE 2 SUMMARY OF RAIL LONGITUDINAL STRESS MEASUREMENT TECHNIQUES

Technique	Comment
Flexural wave propagation	Sensitive to the rail-tie structure damping
X-ray diffraction	Measures surface layer strains only
Acousto-elastic	Sensitive to rail microstructure
Magnetic coercion	Sensitive to rail microstructure and residual stress
Barkhausen noise	Very difficult under field conditions
Electromagnetic-acoustic transducer (EMAT)	Sensitive to rail microstructure and rail surface condition
Laser "spackle"	More useful in lab application due to accuracy required for mapping laser interference patterns

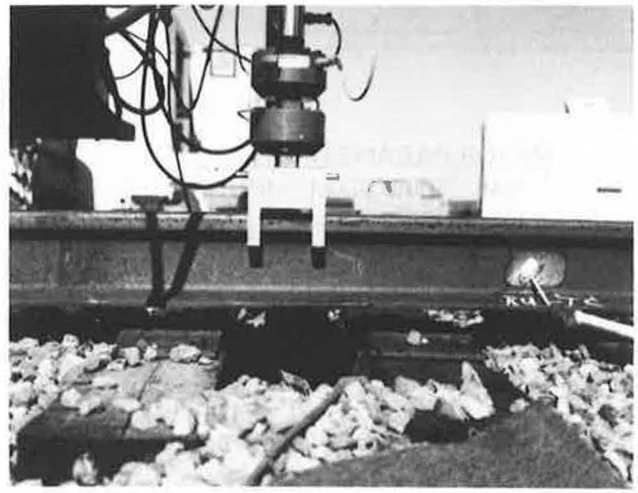


FIGURE 9 Fixture used in the uplift tests.

duce a specified deflection is a measure of these rail forces. The concept implementation is based on the fact that the rail can be conveniently held at the two end points by the wheels of a rail car. This automatically fixes the length of the rail and boundary conditions at the ends of the rail beam. The spikes and anchors between the inner wheels of the two trucks of the car must be removed. Figure 8 shows schematically the rail uplift method; Figure 9 shows the rail-car-mounted hydraulic fixture lifting the test rail.

An analytical model, shown in Figure 10, has been developed to calculate the vertical deflection produced by different levels of rail force. This model proved that the deflection is measurably sensitive within the range of longitudinal forces of interest in buckling safety assessment. Results from the model were used to conduct parametric studies required to

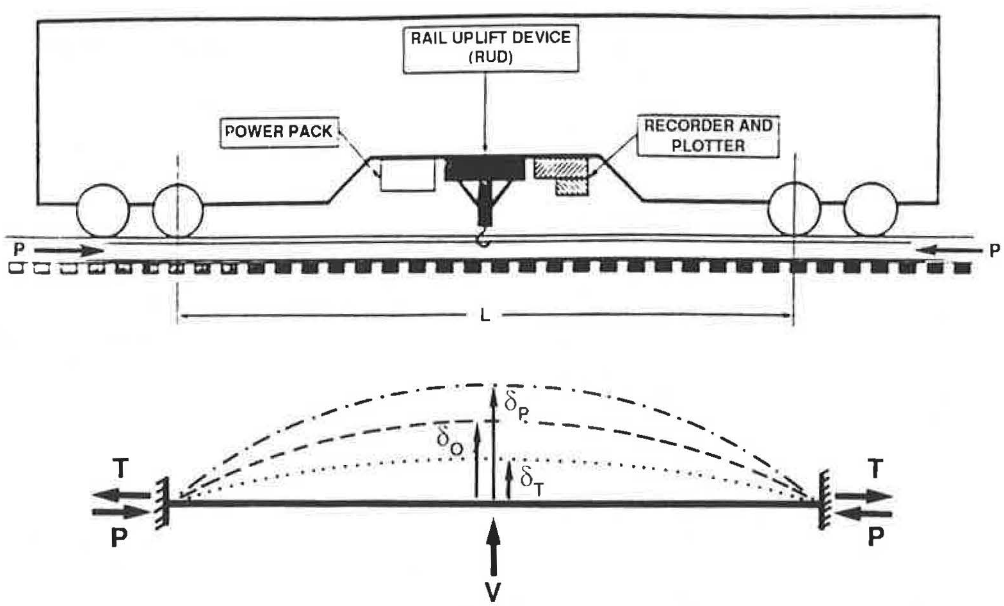


FIGURE 8 Schematic of rail uplift concept.

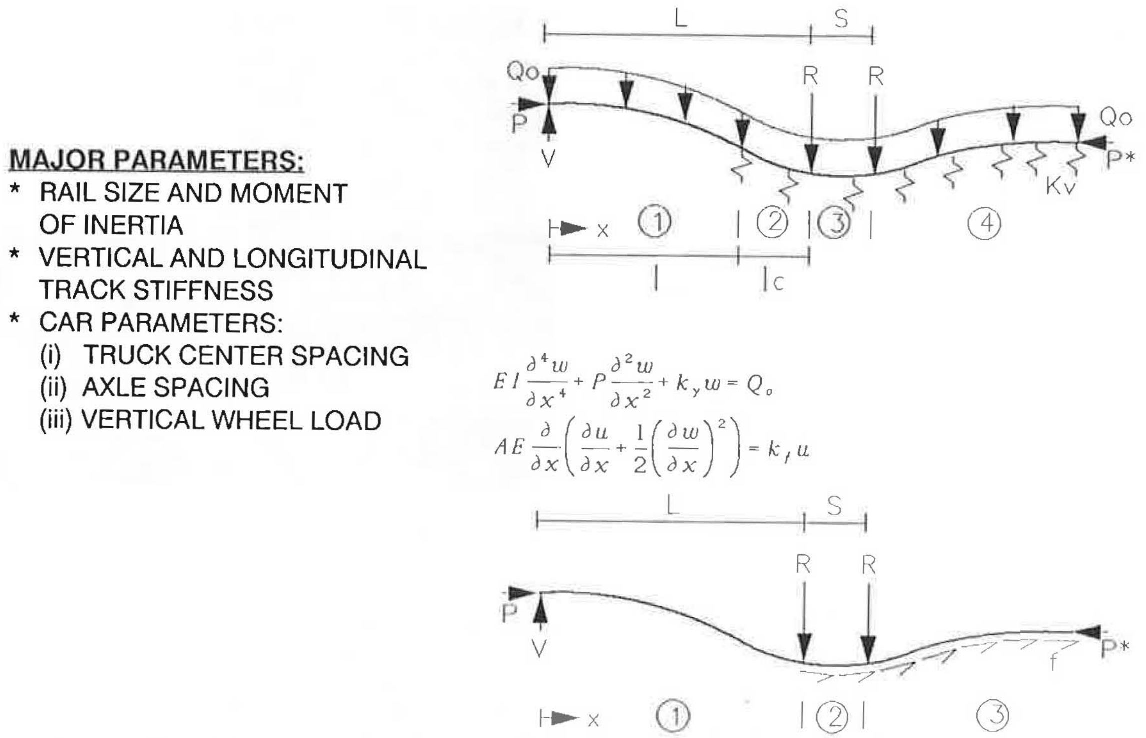


FIGURE 10 Beam bending–rail uplift analysis.

plan the tests, design the test fixture, and assess measurement sensitivity. Figure 11 shows the influence of rail size on the uplift force required for different levels of longitudinal force.

Test Results

Tests were conducted at TTC on a tangent and a 5-degree curved track. A special instrumentation car with inner wheel spacing of 340 in. was adapted to provide a maximum central vertical force of 30 kips. The test sections were instrumented with strain gages, shown in Figure 12, to measure the rail force. The variation in the rail force was achieved by destressing at reasonably high neutral temperatures for tensile loads and by artificial rail heating for compressive force levels. The rail force was correlated with the required vertical load for a 2-in. rail uplift.

Figure 13 shows data on a typical section, which fall on a straight line as theory predicts. Figure 14 shows the regression lines for the eight tested sections of the tangent. From these data the rail force can be determined within an error band of ±12.5 kips. This error is generally tolerable in buckling safety assessment. Figure 15 shows the mean regression line for all the test data and also the theoretical prediction. Agreement between the theory and the test is seen from the figure.

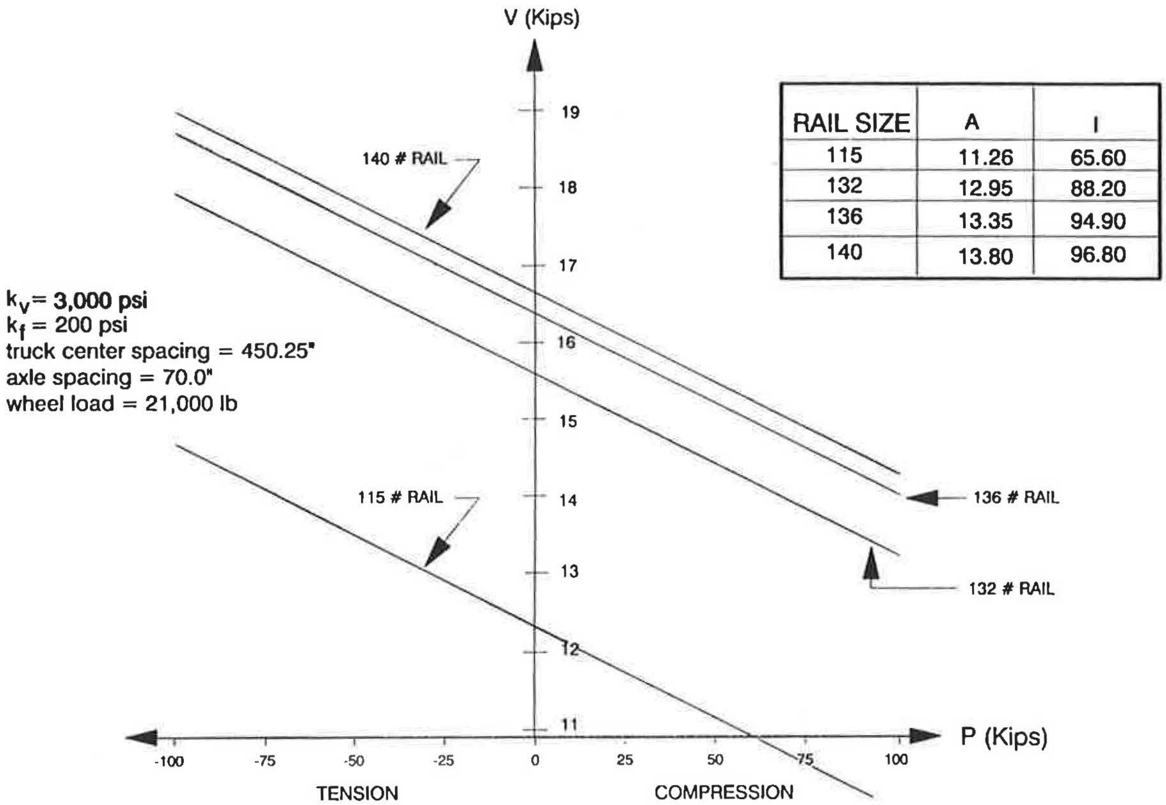
Test data have also been collected on a 5-degree curve. The responses of high and low rails differ from one another and from that of the tangent, as seen in Figure 16. Differences are attributed to the wheel load variations in high and low rails as well as difference in the "effective lengths" of the rail beam under the wheels. Accounting theoretically for these variations resulted in agreement with the recorded data on the curves. Thus, the proposed technique is universal in application and does not need site-specific calibration for curves, provided the superelevation is known. However, this conclusion should be firmly established through additional tests. Some correction may also be needed in cases of excessive rail wear. These and other issues dealing with automated schemes of spike pullout, power pack operations, and measurement of deflection with car-mounted devices will be addressed in a future research program by TSC.

SAFETY ASSURANCE APPLICATIONS

As demonstrated by the authors in another paper in this Record and shown in Figure 17, buckling safety assurance may be attained through appropriate safety criteria of allowable temperature increase (or rail longitudinal force) for various levels of track lateral resistance. Within this framework, the required track resistance can be measured and monitored by the appropriate STPT measurements, and the corresponding allowable rail force determined by a rail car–mounted rail uplift device. This prototype safety assurance concept is undergoing additional research and field implementation studies.

CONCLUSIONS

- Techniques have been developed and prototype hardware is available for the measurement of track resistance and rail longitudinal force (neutral temperature), which, in turn, can indicate incipient buckles or buckling prone conditions. Ad-



RAIL SIZE	A	I
115	11.26	65.60
132	12.95	88.20
136	13.35	94.90
140	13.80	96.80

FIGURE 11 Rail size influence on uplift force versus longitudinal force (605 RFC car, 2-in. deflection).

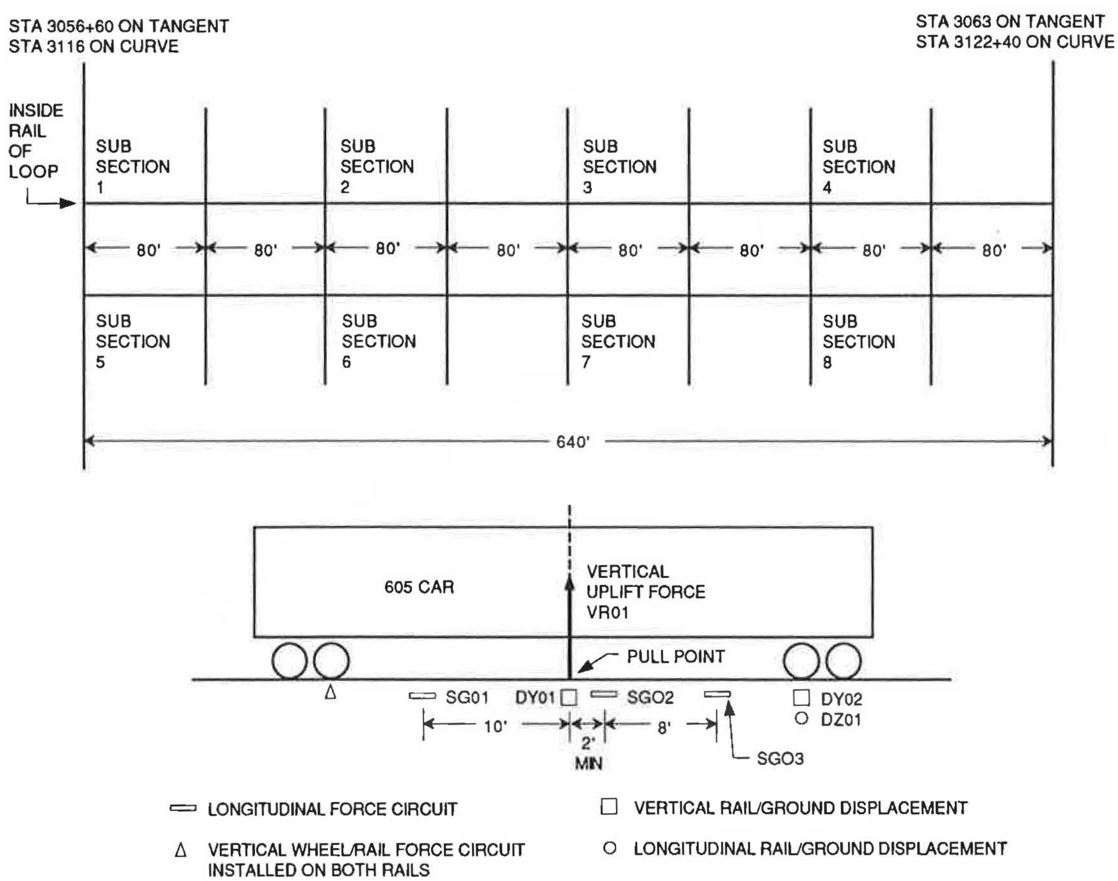


FIGURE 12 Rail uplift test layout and instrumentation.

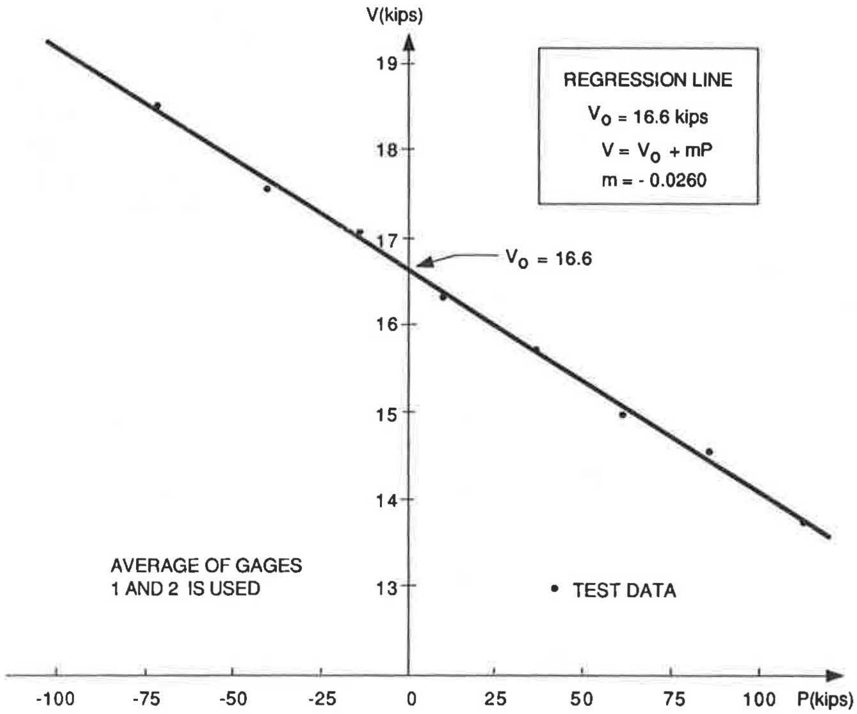


FIGURE 13 Typical uplift force (*V*) versus rail force (*P*) for tangent track (section 4).

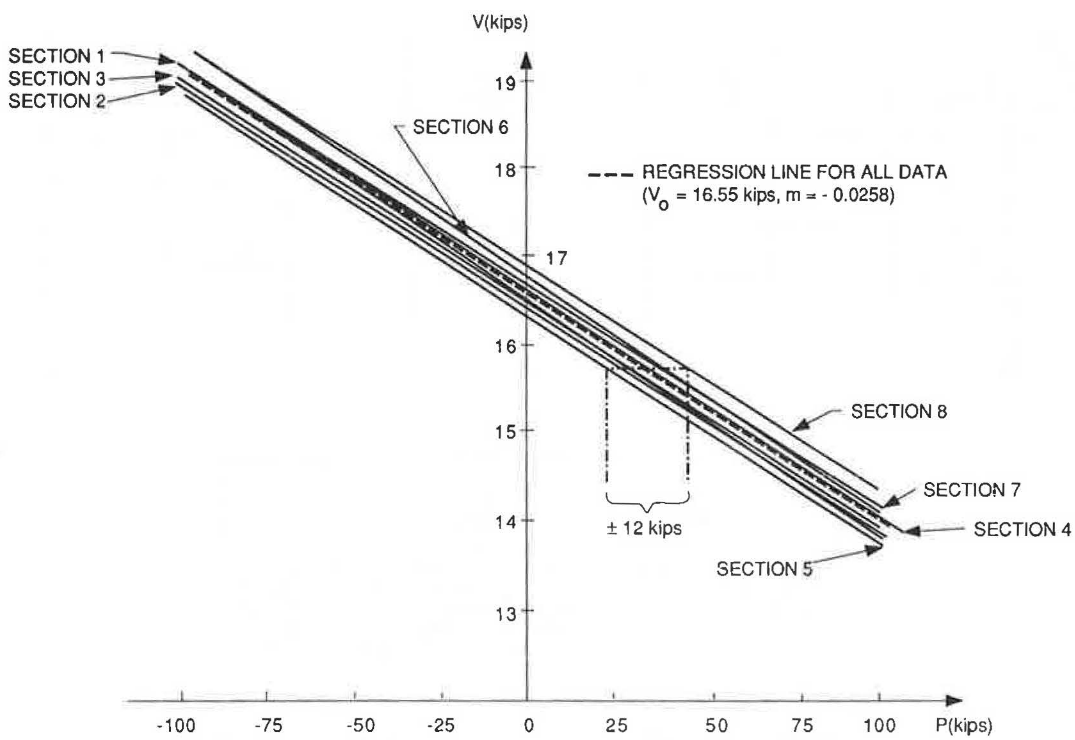


FIGURE 14 *V* versus *P* regression lines for tangent track.

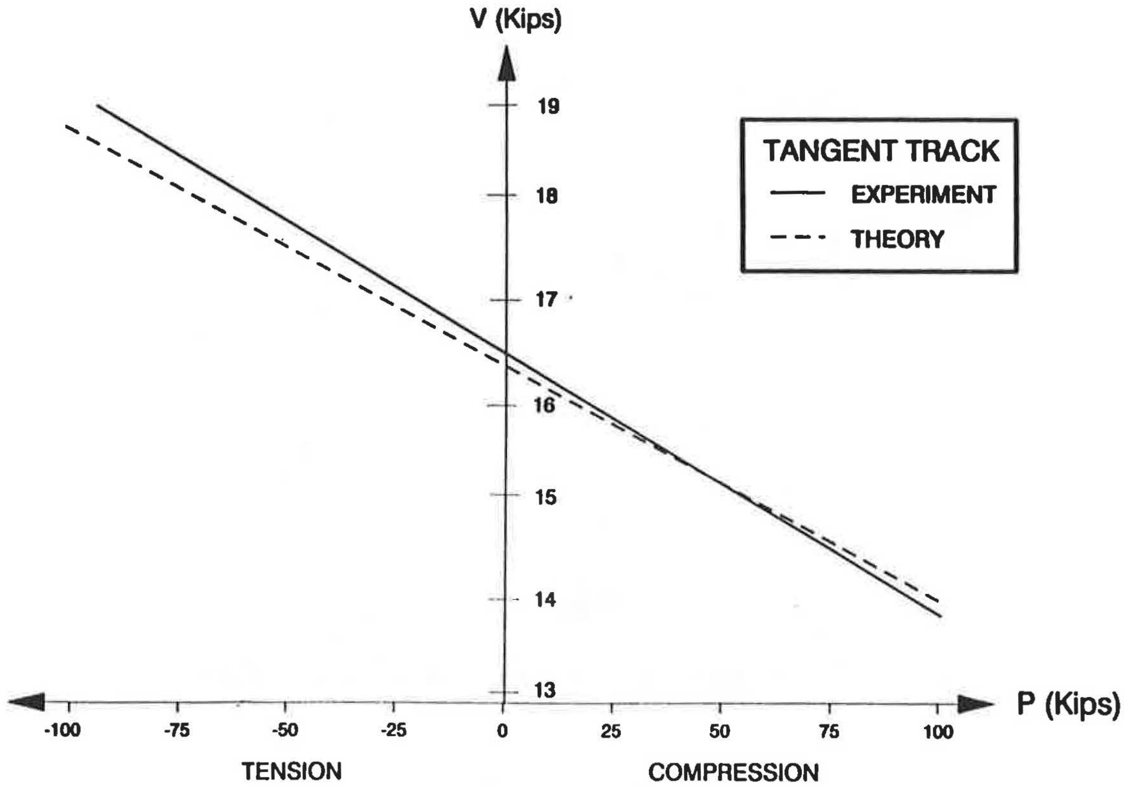


FIGURE 15 Comparison of theoretical and experimental *V* versus *P* behavior.

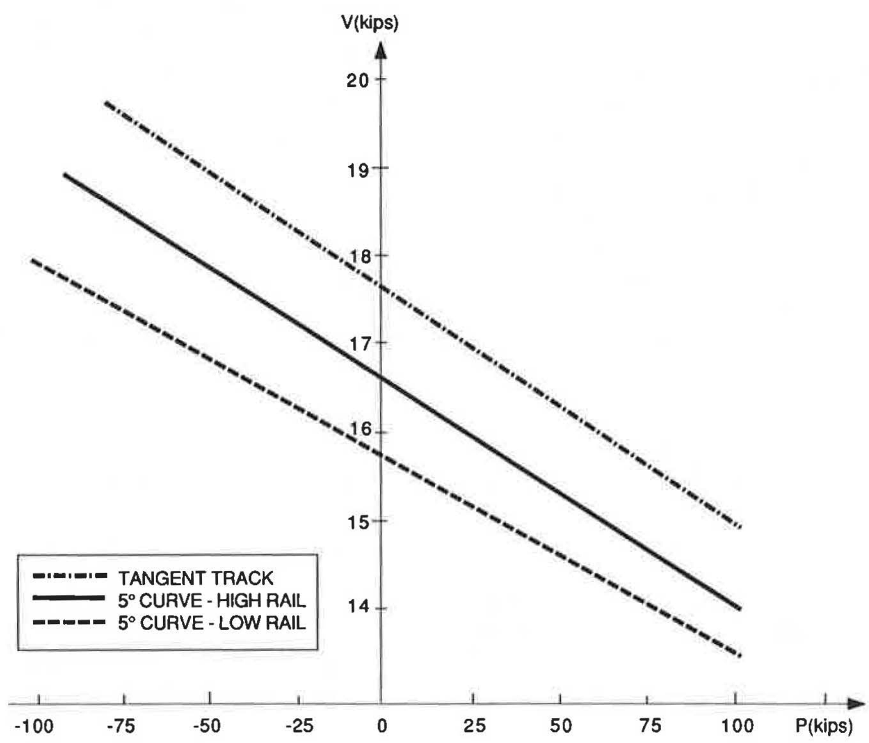


FIGURE 16 Comparison of *V* versus *P* regression lines for tangent and 5-degree curve track.

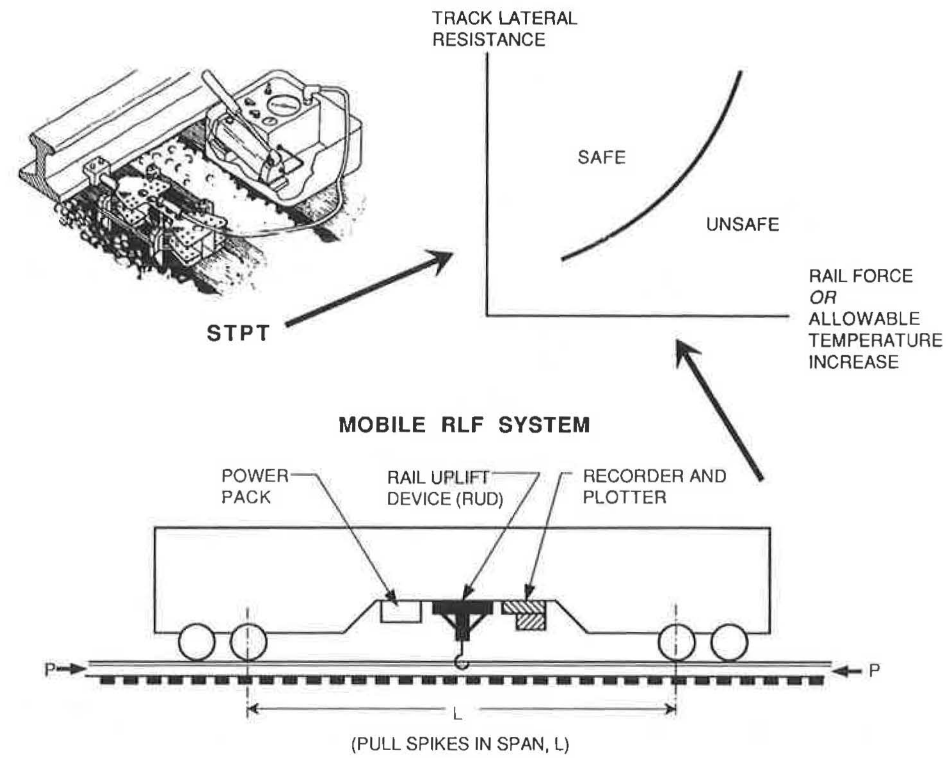


FIGURE 17 Safety limit concept for buckling prevention.

ditionally, these techniques can provide useful tools to guide maintenance activities for improved CWR track safety.

- The track lateral resistance has a nonlinear softening characteristic, on which there are two salient points: a peak value occurring at a fractional lateral displacement, and a limit value at displacements of a few inches. The peak value is sensitive to the consolidation level (MGT). For tamped and weak tracks, the peak and limiting values are very close. The limiting value does not increase at the same rate as the peak value with increased consolidation.
- The STPT device developed is portable and convenient for a quick evaluation of lateral resistance. Both peak and limiting values of resistance can be determined using this device. However, it is adequate to determine the peak value, which involves mobilizing the tie by no more than ¼ in. The limiting value can be estimated by the empirical formulas provided here.
- Although STPT results show scatter, it is usually not severe enough to affect safe buckling safety limit computations. The average of three randomly selected STPT values per 50-ft CWR track segment is adequate for buckling safety assurance of the segment.
- Rail force and hence the neutral temperature can be measured by the rail uplift device developed here. The method is not destructive but requires removal of spikes and anchors under the car. The method yields absolute rail force without site-specific calibration. The accuracy of the method, based on the tests conducted, is within ± 12.5 kips, which is deemed sufficient for buckling safety assurance.

REFERENCES

1. G. Samavedam, A. Kish, and D. Jeong. *Experimental Investigation of Dynamic Buckling of CWR Tracks*. Report DOT/FRA/ORD-86/07. FRA, U.S. Department of Transportation, 1986.
2. A. Kish and G. Samavedam. *Analyses of Phase III Dynamic Buckling Tests*. Final Report DOT/FRA/ORD-89/08. FRA, U.S. Department of Transportation, 1989.
3. G. Samavedam and A. Kish. *Dynamic Buckling Test Analyses of a High Degree CWR Track*. Report DOT-TSC-FRA-85/03. FRA, U.S. Department of Transportation, 1989.
4. A. Kish, G. Samavedam, and D. Jeong. *Influence of Vehicle Induced Loads on the Lateral Stability of CWR Track*. Report DOT/FRA/ORD-85/03. FRA, U.S. Department of Transportation, 1985.
5. D. Jeong, G. Samavedam, and A. Kish. *Determination of Track Lateral Resistance from Lateral Pull Tests*. Report DOT/FRA/ORD-10, FRA, U.S. Department of Transportation, 1983.
6. J. Pietrak, A. Kanaan, A. Kish, and G. Samavedam. *Track Lateral Resistance Test Data Base*. Interim Report DOT/FRA/ORD (in preparation).
7. G. Samavedam and A. Kish. *Track Characterization and Correlations Study*. DOT/FRA report (in preparation).
8. A. Kish, G. Samavedam, and D. Jeong. The Neutral Temperature Variation of Continuous Welded Rails. *AREA Bulletin 712*, American Railway Engineering Association, Washington, D.C., 1987.
9. A. Kish and G. Samavedam. Longitudinal Force Measurement in Continuous Welded Rail from Beam Column Deflection Response. *AREA Bulletin 712*, American Railway Engineering Association, Washington, D.C., 1987.
10. Nondestructive Techniques for Measuring the Longitudinal Force in Rails. (P. Elliott, ed.) Proceedings of a joint government-industry conference, Washington, D.C., Feb. 1979.

Lateral Track Stability: Theory and Practice in Japan

SHIGERU MIURA

In Japan theoretical and experimental studies have been conducted since the early 1930s to work out effective measures to maintain lateral track stability. In 1957 the theory of track buckling based on the principle of virtual work was established, which is the basis for current practical measures to ensure lateral track stability. The theory defines the minimum buckling strength, which corresponds to the minimum longitudinal load at which a stable distortion wave can exist. Based on the theory and practical experience, laying and maintenance standards for continuous welded rail (CWR) and joint-gap control methods have so far been established, both of which have effectively contributed to lateral track stability. Recently, it has become necessary to use CWR even on sharp curves and to remove expansion joints in front of and behind a turnout to reduce maintenance. From this point of view, it is important to make clear the cause of track buckling and to understand more clearly the behavior of long welded rail connected to turnouts. The historical background and the current status of the theory and practice of lateral track stability in Japan are described in this paper.

Thermal longitudinal forces caused by an increase in the temperature of railway track can cause the track to be laterally and suddenly deformed. This phenomenon, called buckling, is sometimes fatal. Its prevention has been a serious concern of track engineers for a long time, especially with the increased use of continous welded rail (CWR), which has been brought into practical use since the 1950s.

In Japan the first theory of track buckling was presented in 1932. In 1957 the theory of buckling was established and is the basis for various measures currently taken to ensure lateral track stability.

The theoretical basis of CWR was defined in 1934, and in 1937 a 4.2-km-long CWR was laid in a tunnel on a trial basis. Thereafter, through experimental verification, regular laying of CWR was started in 1953. By the end of 1983, when the Japanese National Railways (JNR) was still in existence, the total length of CWR laid was about 7940 km, of which 3470 km is on the Shinkansen lines and 4470 km is on the narrow-gage lines and accounts for about 16 percent of the whole length of those lines.

On the basis of theoretical analyses and practical experience on the buckling stability of track, laying and maintenance standards for CWR and a joint-gap control method have been established in Japan, both of which have been effective contributors to the prevention of track buckling. The theory and practice of lateral track stability of the now-defunct JNR and the Japan Railways (JR) Group, which took over after privatization of JNR, will be described here.

Track and Structure Laboratory, Railway Technical Research Institute, 2-8-38 Hikari-cho, Kokubunji-shi, Tokyo 185 Japan.

THEORY OF LATERAL TRACK STABILITY

Minimum Buckling Strength

It was not until 1932 that studies on lateral track stability were undertaken in Japan. Horikoshi had carried out buckling tests on a full-scale test track fixed with concrete blocks at both ends of a 48-m-long track. In 1934, on the basis of the test results, he established a theoretical equation for track buckling. Around the same time, Inada of Kyushu Imperial University studied railway track buckling as a part of the stability theory of a long column subjected to lateral elastic resistance. Further, in 1938 Hoshino established an expansion and contraction theory of CWR on the basis of expansion tests on a full-scale track and its theoretical consideration. In 1943 Ono derived buckling loads from a differential equation in which the ballast resistance was assumed constant and the balance of a longitudinal rail force in front of and behind a buckling waveform was taken into consideration.

Thereafter, Numata suggested a new buckling theory (*1*) based on the principle of virtual work. This theory is a foundation for the various countermeasures currently taken in Japan against track buckling and is outlined as follows.

The shapes of CWR that are subjected to longitudinal force and laterally buckled are categorized as shown in Figure 1 and approximated with a sinusoidal waveform. Meanwhile, ballast resistance should be constant regardless of displacement, as shown in Figure 2c. Here, the following energies accumulated in a track are taken into consideration:

(a) Strain energy generated by longitudinal force change,
(b) Strain energy generated by rail bend, and
(c) Internal energy created by ballast resistance.

An application of the principle of virtual work to these energies yields the following expression of buckling strength:

$$P_t = P + \left\{\frac{\gamma^2 r^2}{P} + \frac{\alpha r}{P^3 (P)^{1/2}}\left[\left(g - \xi\frac{P}{R}\right)^2 + \kappa\left(g - \xi\frac{P}{R}\right)\frac{P}{R}\right]\right\}^{1/2} - \frac{\gamma r}{(P)^{1/2}} \quad (1)$$

where

P_t = buckling strength,
P = longitudinal rail force balanced after buckling,
g = longitudinal ballast resistance,
r = lateral ballast resistance,

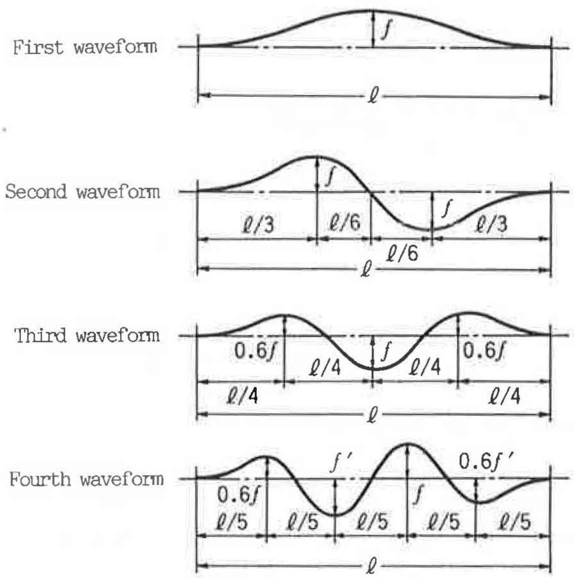


FIGURE 1 Classification of buckling waveforms.

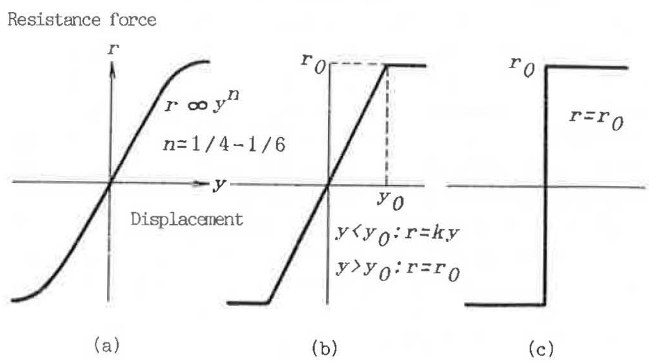


FIGURE 2 Relation between tie displacement and ballast resistance.

R = radius of track curvature, and
γ, α, ξ, κ = constants that depend on track structures and waveforms.

As for virtual wavelength (l) and displacement (f), the buckling strength determined by the above equation and the relationship between l and f in balance after buckling are shown in Figure 3. The longitudinal force less than the minimum value of buckling strength, as shown by Equation 1 and Figure 3, does not generate buckling.

When minimum buckling strengths by radii of curvature are determined for the various waveforms shown in Figure 1, on tangent track and track with a larger radius of curvature the minimum buckling strength of the second waveform is the smallest, whereas on the track with a smaller radius of curvature the minimum buckling strength of the first waveform is the smallest. The minimum buckling strengths determined for various ballast resistances are shown in Figure 4.

The minimum buckling strength shown in Figure 4 sets a limit at which the longitudinal force less than the one corresponding to the minimum buckling strength cannot bring

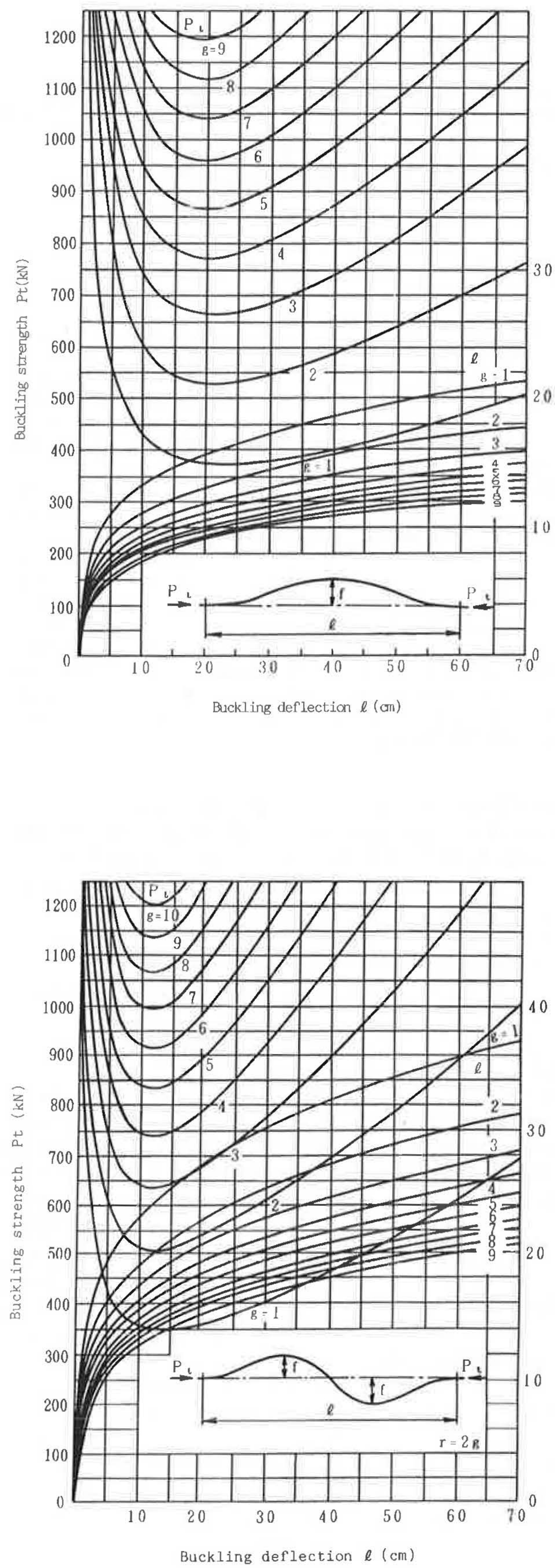


FIGURE 3 Buckling strength of track.

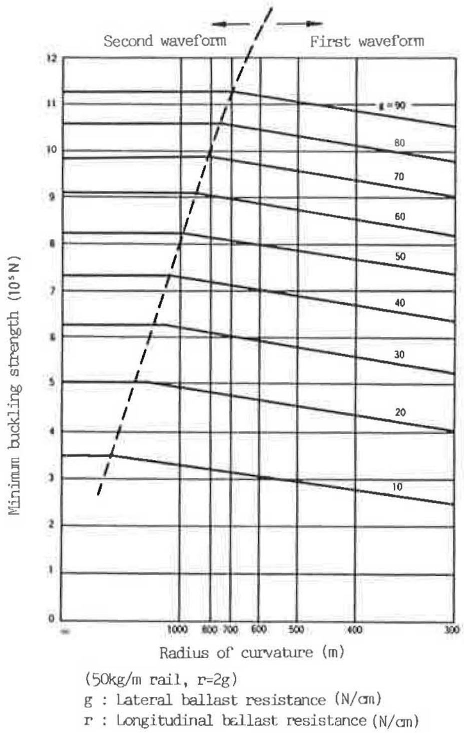


FIGURE 4 Minimum buckling strength as a function of radius of curvature.

about any balance under the condition of bent rail. However, the minimum buckling strength does not represent the load that actually induces buckling.

In order to make clear the relation between the minimum buckling strength and the load that actually induces buckling, 400 model tests were carried out on tangent and curved tracks. The loads that induced buckling in the tests were found to be distributed in an approximately normalized form where theoretically calculated minimum buckling strength constituted a lower limit. Here, the relationship between longitudinal force at the time of buckling and lateral displacement of track panel is in agreement with the theoretical calculations. The variation of the loads that induced buckling was caused by track irregularity, variation of lateral ballast resistance, and the like. Furthermore, the results of buckling tests on full-scale track in 1957 demonstrated the theory's validity.

As stated above, Equation 1 gives the lowest magnitude of loads that induce buckling and has a certain margin to the longitudinal force that causes an actual buckling. The margin depends on the variation of ballast resistance, uneven lift of track panel, initial irregularity, and other factors. Thus, in practical application of Equation 1, 70 percent of measured ballast resistance is adopted, considering the variation of ballast resistance, uneven lift of track panel, and so on. Buckling stability also is examined, allowing for a margin of 20 percent for the longitudinal rail force determined by Equation 1. Thus a sufficient safety factor can be guaranteed. Because the equation is a little complicated, the following simplified equation, which yields a good approximate value, is preferred:

when $R \geqslant R_0$,

$$P_{t2}^* = 3.63\ J^{0.383} g^{0.535} N_j^{0.267} \quad (2)$$

when $R < R_0$,

$$P_{t1}^* = 3.81\ J^{0.383} g^{0.535} N_j^{0.267} - 20.2\ j^{0.789} N_j^{0.600}/R \quad (3)$$

where

$R_0 = (112.2\ J^{0.406} N_j^{0.333})/g^{0.535}$,
P_{tn} = track buckling strength (tf) expressed by the buckling waveform with the number of waves (n),
J = lateral rigidity,
g = lateral ballast resistance (kgf/cm),
r = longitudinal ballast resistance (kgf/cm),
N_j = flexural rigidity of track panel (including lateral rail rigidity and multiples of it), and
R = radius of curvature (m).

Buckling Analysis Using Energy Method

Thereafter, in order to give a theoretical basis to the actual buckling generating load, a theoretical analysis using an energy method was carried out (*2*). It is summarized as follows:

- As track deformation caused by buckling occurs, the first and second waveforms in Figure 1 are assumed.
- Lateral track resistance force (g) is expressed in Figure 5 as follows:

$$g = g_0 y/(y + a) \quad (4)$$

- Longitudinal ballast resistance is constant regardless of displacement.
- The rotating resistance moment is expressed by the formula

$$\tau = \tau_0\ (\theta)^{1/2} \quad (5)$$

where τ_0 is a constant and θ is the angle of rotation.

- Longitudinal rail force after buckling is shown in Figure 6.

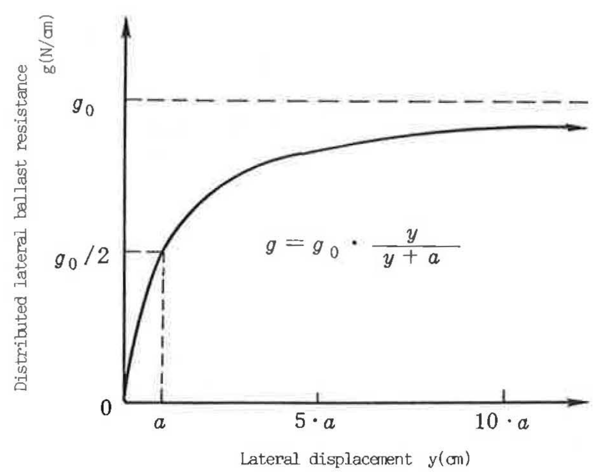


FIGURE 5 Characteristics of lateral ballast resistance.

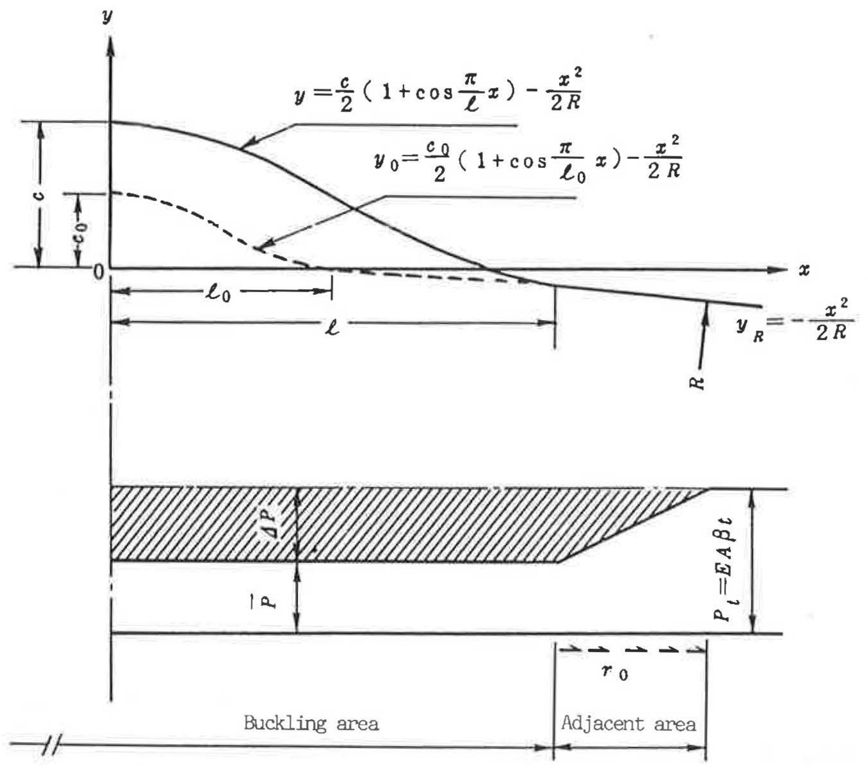


FIGURE 6 Assumption of deformation form and longitudinal force distribution (first waveform).

By virtue of the above, rail-axial strain energy, rail-bend strain energy accumulated in rail, work done against lateral and longitudinal ballast resistance, and work done to overcome the rotating resistance of rail fastening are determined.

When, on a track with initial irregularity, the total of the above-mentioned energies and of work done for track deformation wavelength (l) and lateral displacement (c) caused by a temperature rise (t) is equal to ΔU, this value has extremes depending on c and l, and the stability of deformation can be judged by these extremes. By fixing the lateral displacement magnitude c and partially differentiating with l, the minimum value of ΔU can be determined. The relationship between these ΔU and c is shown in Figure 7. A portion of the diagram with small values of c is on a linear scale, whereas the rest of the diagram with larger values of c is on a logarithmic scale.

The minimum value of ΔU indicates a stable balance, and the maximum value represents an unstable balance. Figure 7 shows that, when t is less than 40°C, the balance is stable only against minute displacements; when t is equal to 49°C, separate inflection points come out between C = 10 cm and C = 20 cm; when the temperature is higher than 49°C, a distinct minimum value appears within a larger displacement range. In other words, a stable balance generates in this range. When the temperature rises, the minimum value, or balance, can no longer be found in a minute displacement range—it exists only within a larger displacement range. The relationship between this minimum value and temperature variation t is shown in Figure 8, in which continuous lines indicate stable balance, and broken lines represent unstable balance.

Figure 8 shows that with less variation of temperature, a stable balance appears only under a minute deformation, whereas at a temperature exceeding a certain degree a stable balance emerges under a larger deformation as well as under a minute deformation. The longitudinal rail forces corresponding to the temperature variations coincide with the minimum buckling strength as described above. It is seen also from Figure 8 that, even with the temperature variation that exceeds the one corresponding to the minimum buckling strength, a balance state under a minimum deformation exists and does not immediately lead to a larger deformation.

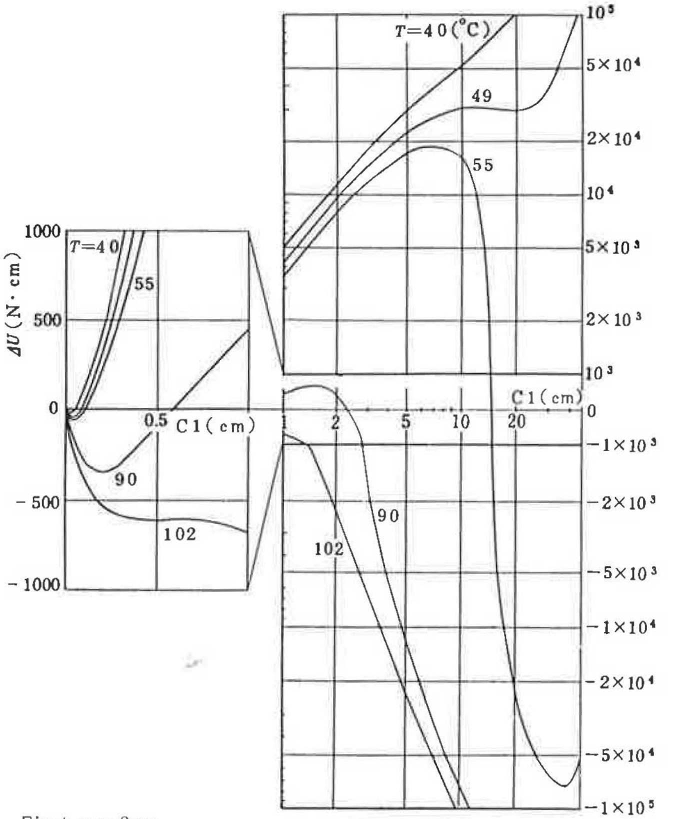
The results of these analyses are as follows:

- An ultimate lateral ballast resistance considerably influences the maximum longitudinal force (Load A), under which a stable balance can be kept under a minute deformation and the minimum buckling strength (Load C).
- Initial characteristics of a lateral ballast resistance greatly influence Load A.
- A longitudinal ballast resistance has a great effect on Load C, but a small one on Load A.
- Influences of rotating resistance generated from rail fastenings are small in general.
- Alignment and inital track irregularity greatly influence Load A, but slightly influence Load C.

TESTS OF LATERAL TRACK STABILITY

Characteristics of Lateral Ballast Resistance

The characteristics of lateral ballast resistance have a great influence on buckling strength of track. Therefore, in order



First waveform
R= ∞
50N Type rail (50.4kg/m)
Ultimate lateral ballast resistance form go=40N/cm
Initial characteristic coefficient of lateral ballast resistance a=0.1cm
Longitudinal ballast resistance ro=60N/cm
Rotating resistance moment of rail fastening device ro=5000N
Initial irregularity of alignment Maximum irregularity Co=2cm
Irregularity wavelength 2lo=10cm

FIGURE 7 Relation between energy variation and lateral displacement.

to evaluate lateral track stability, it is important to define the characteristics of ballast resistance. Lateral ballast resistance depends not only on dimension, geometry, mass, and spacing of ties but also on profile, bulk density, compacting magnitude of ballast, and so on. Figure 9 shows the characteristics of lateral ballast resistance that resulted from the tests with ties laterally pulled on the track under commercial operation. These tests have revealed that the characteristics of lateral ballast resistance are expressed by a hyperbola with a good approximation; lowered ballast resistance by tamping is restored in due course by train running; and so forth. In the meantime, as a result of the tests with ties laterally pulled on a test track, it has been ascertained that the lateral ballast resistance per tie is expressed by the following equation:

$$F = aW + brG_e + crG_s \tag{6}$$

where

F = ballast resistance per tie,
W = track mass on a tie,
r = bulk density of ballast,
G_e = statical moment of area around top chord of a tie end,
G_s = statical moment of area around top chord of tie side face, and
a, b, c = coefficients in Table 1.

Moreover, these tests have revealed that the tie bottom, side, and end surfaces share a third of the resistance with one another.

Buckling Tests

Several buckling tests, including the ones by Horikoshi as described above, were carried out on full-scale tracks in Japan.

The tests in 1932 were performed not only on a tangent track but also on curved tracks with radii of curvature 300 m and 500 m constructed on a 48-m-long test track. Longitudinal force was applied to the rail by means of hydraulic jacks and vapor pipe heating. The tests in 1956 were carried out on a 320-m-long test track with a 600-m curve radius. Longitudinal force was applied by vapor pipe heating, yielding data such as buckling wavelength and buckling length. In 1964, before the inauguration of the Shinkansen, on several sections of its line under varied ballast conditions, the rails were heated to buckling in trials with acetylene gas burners, verifying their safety against buckling.

Thereafter, beginning in 1981, a new buckling testing unit was installed on a full-scale track at the Railway Technical Research Institute, and seven series of various tests were performed, along with a study on a buckling stability theory. A test using this unit is shown in Figure 10. On a test section approximately 60 m long, the tests on tangent sections, curved track with radii of 300 m or less, and turnouts can be conducted. As for rail heating, a temperature rise to 70°C can be generated within 60 min with a flow of direct current through the rail.

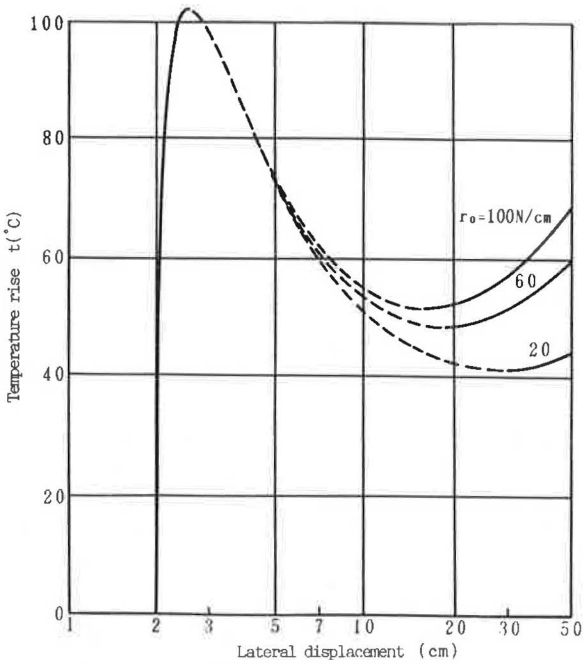
The tests performed until now using this testing unit are as follows: buckling tests on a tangent section and curved sections with radii of curvature less than 400 m, buckling tests on wooden tie track, buckling tests that take the effect of load on the track into consideration, tests on longitudinal force characteristics of turnouts, and buckling tests on two tracks with different gages laid side by side. As a result of these tests, it was made clear that the value of a buckling-generating load on normal tracks is between Load A and Load C, determined by the theoretical analysis discussed previously. However, it is necessary to continue the investigation into the quantitative relationship between various factors and the buckling-generating load. The results of tests on sharp curve sections and turnouts have been implemented in engineering practices.

PRACTICES IN TRACK BUCKLING STABILITY

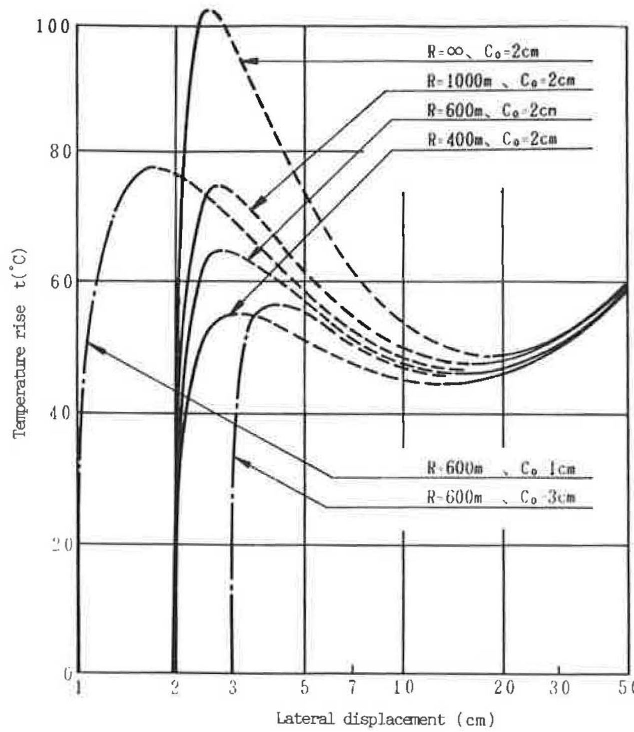
Laying and Maintenance of CWR

For lateral track stability, the track conditions for laying CWR in Japan were established as follows:

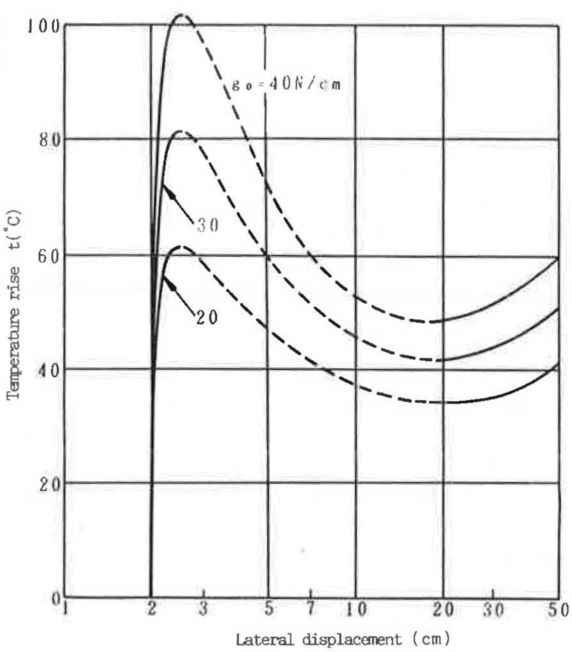
- For the rail with a mass of 50 kg/m or more, the number of ties must be more than 38 per rail unit length of 25 m;



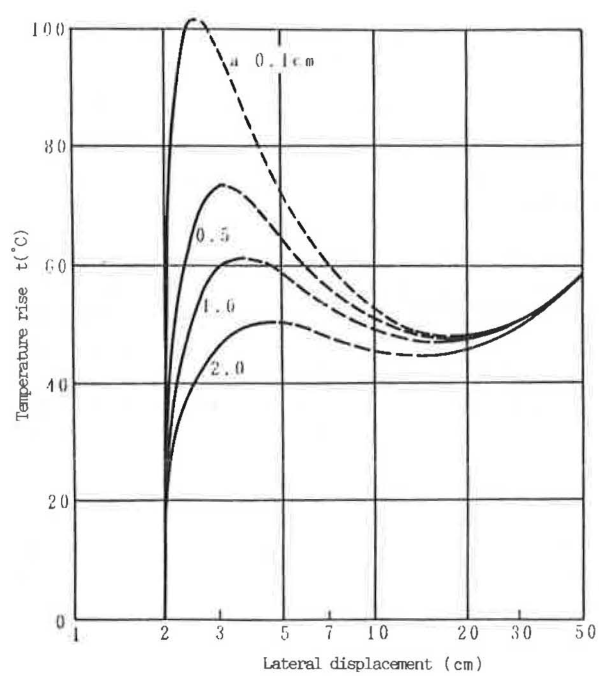
(A) Influence of Distributed Longitudinal Ballast Resistance (ro)
(First waveform)



(B) Influence of Alignment and Initial Track Irregularity
(First waveform)



(C) Influence of Ultimate Distributed Lateral Ballast Resistance (go)
(First waveform)



(D) Influence of Initial Characteristic (a) of Distributed Lateral Ballast Resistance (First waveform)

FIGURE 8 Relationship between temperature rise and lateral displacement.

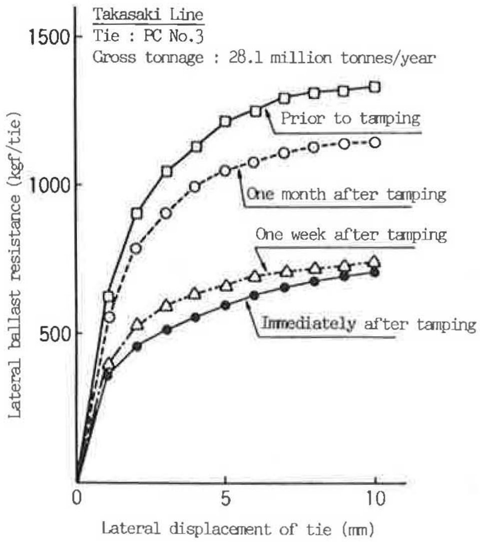


FIGURE 9 Characteristics of lateral ballast resistance.

TABLE 1 COEFFICIENTS FOR VARIOUS BALLASTS

Coefficients	a	b	c
Concrete tie and crushed stone ballast	0.75	29	1.8
Wooden tie and crushed stone ballast	0.75	29	1.3
Wooden tie and gravel ballast	0.6	29	1.4

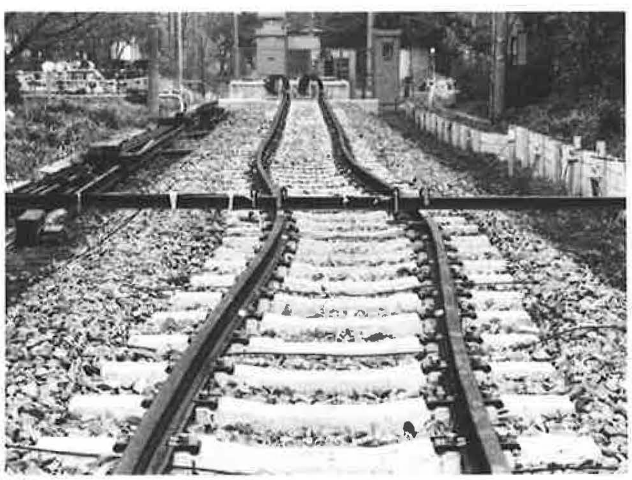


FIGURE 10 Full-scale test of track buckling.

- For track alignment, the radius of curvature must be 600 m or more, and the vertical curve radius at a changing point of gradient must be 2000 m or more;
- The road bed must be stable and free from subsidence;
- The ballast must consist of crushed stone;
- The ballast shoulder must be 400 mm or more wide;
- The lateral resistance must be kept to 4.0 N/mm or more for the 50-kg/m rail, and 5.0 N/mm or more for the 60-kg/m rail.

Meanwhile, on the Shinkansen lines that were constructed of CWR for high speeds of more than 200 km/h, it is specified that ballast resistance be more than 9.0 N/mm on the standard sections and more than 1000 kg/m on the sections subjected to additional force at bridge ends. Furthermore, the tightening temperature of CWR in general must be within the range shown in Figure 11.

A routine control of CWR track to prevent buckling is carried out such that its tightening temperature, creepage, work history at low temperature, and ballast conditions are grasped, which enables comprehensive decision making about the buckling stability of the track before the planning and implementation of CWR tightening changes, ballast maintenance, and so on. A flow chart depicting this process is shown in Figure 12.

To be more precise, when the tightening temperature is less than specified, or when work on rail renewal or on loosening and tightening of rail fastenings on a considerably long section is undertaken at a low temperature, or when creepages are different in different portions of a certain CWR, longitudinal rail force in the summer is greater than that of the standard CWR. A reduced additional temperature is determined by converting this additional longitudinal force into temperature difference, whereas a ratio of ballast resistance for the standard state is obtained from sectional geometry of ballast, which yields a safety factor through the following equation:

$$\alpha = 1.2 i^{0.535}/(1 + \Delta t/\Delta t_{max}) \qquad (7)$$

where

α = safety factor of CWR,
i = ratio of lateral ballast resistance,
Δt = reduced additional temperature, and
Δt_{max} = regularly allowable rate-of-rise from a tightening temperature.

The safety factor defined by the above equation is the ratio of the minimum buckling strength described previously to the maximum longitudinal rail force, including added longitudinal force. Depending upon this value, the necessity of tightening changes or ballast maintenance is decided.

Joint Gap Control on Jointed Track

The joint gap on jointed track must be maintained through periodic inspection, judgment, and alignment to ensure lateral track stability.

The judging standards for lateral track stability of rail joint gaps are categorized administratively in three ranks, according to the ratio of the maximum longitudinal force (P) on jointed track to the minimum buckling load (P_t) described previously. Here, the maximum longitudinal force (P) on jointed track is determined by the following equation:

$$P = EA\beta\,(t_{max} - t - e/\beta l) + R_0 \qquad (8)$$

where

P = possible maximum longitudinal force,
E = Young's modulus for rail steel,

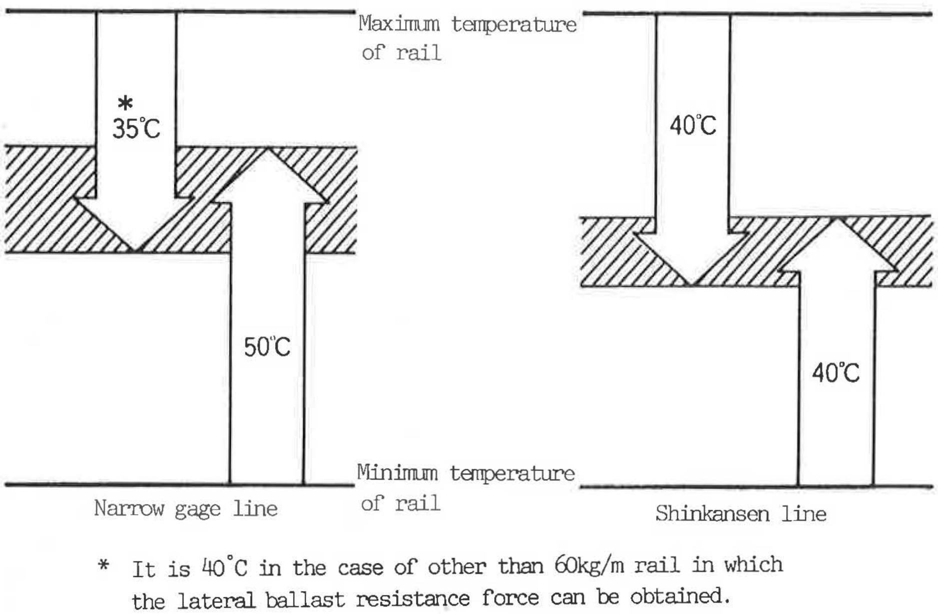


FIGURE 11 Tightening temperature of CWR.

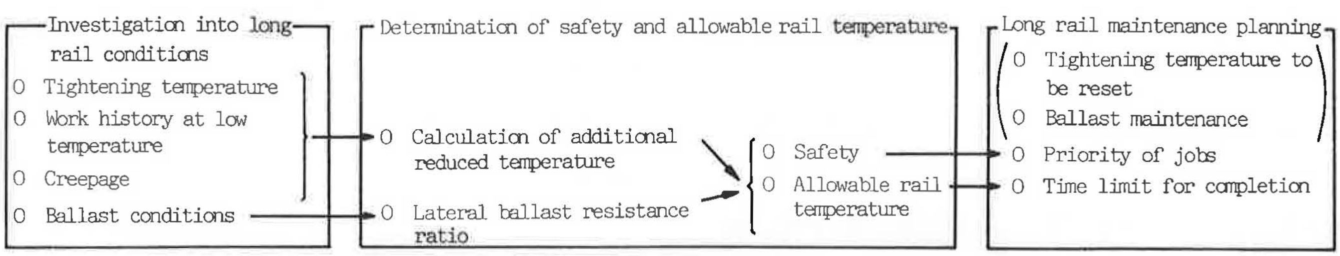


FIGURE 12 Concept of CWR maintenance.

β = coefficient of linear expansion for steel,
t_{max} = possible maximum rail temperature,
t = rail temperature at inspection,
e = rail joint gap at inspection,
l = rail length, and
R_0 = restraining force of joint bar.

RECENT STUDIES

CWR Use on Sharp Curves

CWR use in Japan so far has been limited to curve sections in which the radii of curvature exceed 600 m. The reason is that the volume of railway traffic in Japan is enormous and the frequency of rail renewal because of wear is high on sharp curve sections; the size of ties on the narrow gage lines is so small that the lateral ballast resistance is not sufficiently maintained, which leads to a lower safety against buckling. However, in order to fully exploit excellent features of CWR, it recently has been considered necessary to extend its use to curve sections in which the radii of curvature are smaller than 600 m. The study for implementing this idea is currently being undertaken.

According to the conventional theoretical analysis described previously, the effect of the radius of curvature on the minimum buckling strength is insignificant. Consequently, so long as buckling stability is evaluated in terms of the minimum buckling strength as in recent practice, CWR use on sharp curves should offer no serious problem. However, in practice, when safety on a sharp curve section is evaluated by means of the conventional method, it is feared that the real safety factor is lowered. Therefore, on the basis of the recent track buckling theory an investigation into its quantitative evaluation is being made.

Figure 7, expressing the variation of energy and workload induced by lateral track deformation, gives useful information pertaining to this problem. According to this figure, under a temperature variation of 49°C, which approximately corresponds to the minimum buckling strength (Load C), the value of ΔU corresponding to the balance state at major deformation (C = 20 cm) is at a higher level than the ΔU value at minor deformation (C = 0.1 cm or less). Here, in order to keep a balance state at major deformation, it is necessary to supply energy from the outside or to do work equivalent to the difference between these two ΔU values. Accordingly, it is likely that the difference in ΔU values between the two balance states has something to do with suppressing the buckling. Thus, the relationship between ΔU values and radii of curvature is shown in Figure 13. It is evident from the figure that these values are considerably variable, depending on radii of curvature. From this fact, it seems that the minimum buckling strength does not much depend on the radius of curvature, while the margin to buckling is substantially lowered as the radius of curvature decreases. The quantitative relationship between ΔU value difference and safety factor to buckling as well as the relationship of ΔU values versus various factors lowering the buckling load on real track and their compensation must be investigated further.

CWR Connected to Turnouts

The connection of CWR to turnouts was tried early on the German Federal Railways (*3*). In Japan such an attempt was not made until recently except in experimental cases. Instead, expansion joints usually were located in front of and behind a turnout. One of the problems of direct connection of a turnout to CWR by welding or glueing is the increased longitudinal force generated near the point of the turnout by the two tracks being joined there.

In order to solve this problem, full-scale tests were carried out, complemented by a theoretical analysis (*4*). The outline is as follows:

- The model of a turnout track used is one shown in Figure 14.

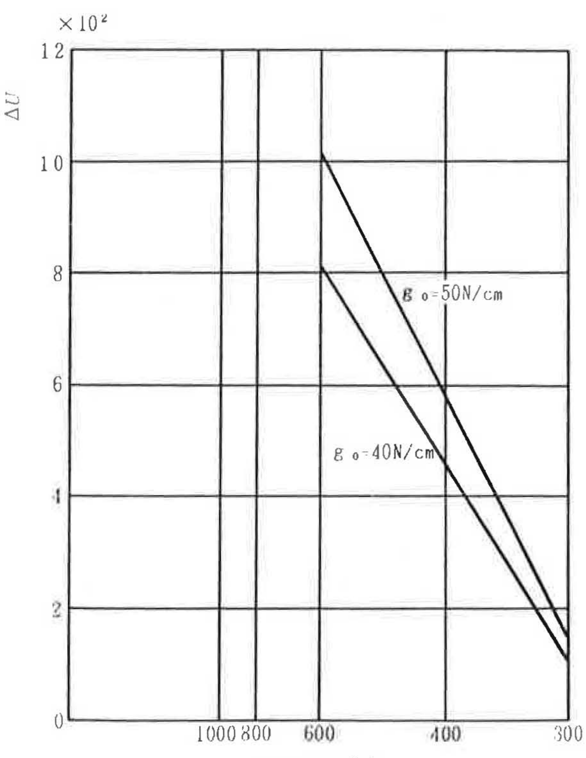


FIGURE 13 ΔU as a function of radius of curvature.

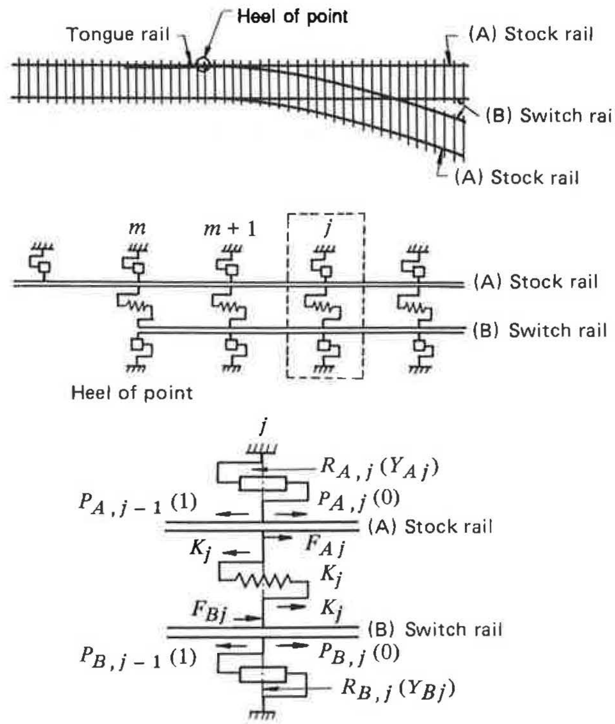
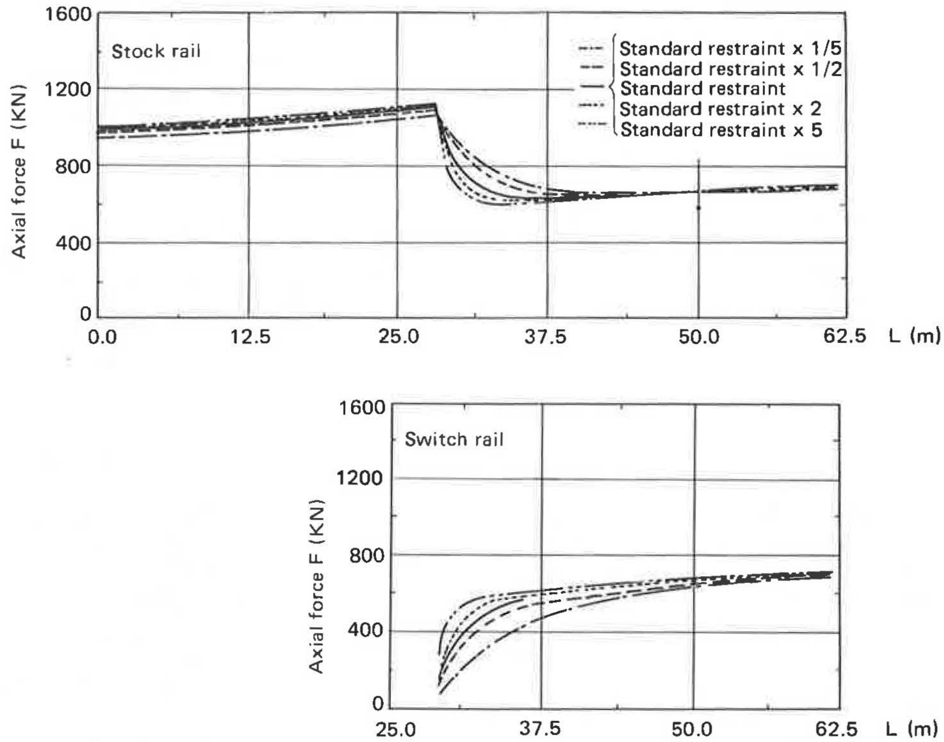


FIGURE 14 Dynamic model of turnout.

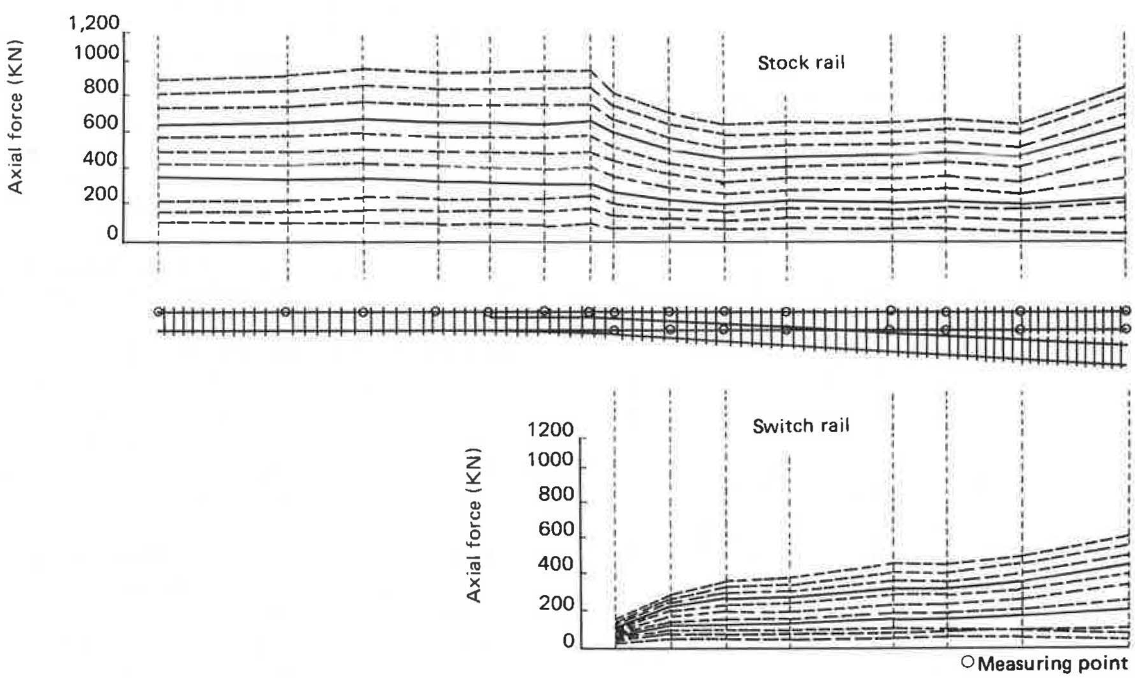
- Provided stock rail and lead rail are connected to each other through a spring system, the spring constant used is one obtained from full-scale tests.
- Characteristics of longitudinal ballast resistance are similar to the ones in Figure 5.

Under the conditions mentioned above, a computer simulation of the change of rail expansion and longitudinal force caused by temperature change was done. A comparison of the simulation results with measurements is made in Figure 15, which shows that the longitudinal rail force changes near the turnout, with its maximum value generated within the stock rail near the heel. The maximum value of the longitudinal rail force for the turnout rail is larger than that for standard CWR. Figure 16 shows the rate of longitudinal rail force increasing with parameters such as longitudinal ballast resistance and rail restraining spring constant. Comparison between the results of the above-mentioned analysis and full-scale tests has revealed the following:

- The results of analysis of longitudinal rail force agree well with analytical results of the full-scale tests.
- The maximum value of longitudinal rail force near the turnout generates near the heel portion of the turnout. The value is about 1.35 times the value of rail axial force in standard CWR.
- The restrained spring constant between rails and longitudinal ballast resistance influences the distribution of longitudinal rail forces but influences slightly its maximum value.
- Within about 30 m of the heel, longitudinal rail force is larger by 5 percent or more than the longitudinal force in



(A) Calculation



(B) Full Scale Test

FIGURE 15 Longitudinal force distribution of turnout.

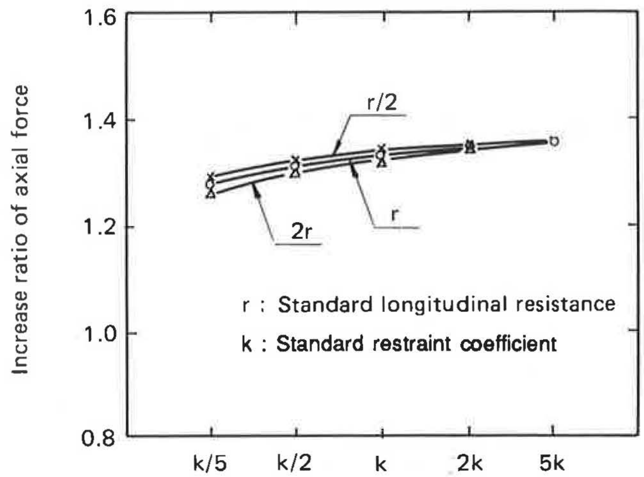


FIGURE 16 Variation of the maximum longitudinal force depending on *k* and *r*.

standard CWR under the same temperature variation as in the turnout rail.

- There is relative displacement between the stock rail and the point rail.

Owing to the above facts, when CWR is connected with the turnout, measures to increase ballast resistance at the 20- to 30-m-long portion of the turnout from the heel to the tip of the point rail should be taken. In addition, a means to prevent a large relative displacement between the stock rail and the point rail will be adopted. On this line, applications of CWR connected with turnouts are now advancing to the practical stage.

CONCLUSIONS

In Japan all the lines of the JR Group and most of the lines of private railways except for the Shinkansen with standard-gage track and some of private railways are on the 1067-mm-gage tracks. This gage has many disadvantages with respect to lateral track stability. It is difficult for this gage to hold sufficient lateral stability because the size of the ties and the mass of the ballast are smaller than those for the standard gage. Moreover, Japanese topography features mountainous terrain and hence many steep curves and gradients along the railway lines. Nevertheless, the railway traffic volume in Japan is considerably higher in comparison with foreign railways. Also, most of the traffic is generated from passenger operation. Consequently, it is very important to keep the track in good condition and to secure its lateral stability. It is believed that efforts so far and maintenance practices established on the basis of the results have been considerably successful, but the efforts are expected to be continued to make further advances.

REFERENCES

1. M. Numata. Buckling Strength of Continuous Welded Rail. *Bulletin, International Railway Congress Association*, 1960, pp. 33–49.
2. T. Miyai. Numerical Analysis of Track Buckling by Energy Methods. *Quarterly Report of RTRI*, Vol. 26, No. 3, 1985, pp. 81–84.
3. J. Eisenmann. Theory and Practice of the Continuous Welded Rail. *Railway Technical Review*, No. 27, 1985/86, pp. 35–39.
4. S. Miura and H. Yanagawa. Characteristics of Axial Force in Rail at Turnout Integrated with Continuous Welded Rail. *Quarterly Report of RTRI*, Vol. 30, No. 4, 1989, pp. 202–206.

Union Pacific's Approach To Preserving Lateral Track Stability

William C. Thompson

The discussion is focused entirely on the in-track behavior of continuous welded rail (CWR) as it affects the lateral stability of railroad track. Lateral track stability is conditioned by the interrelated actions of the various elements of the vehicle and track system [various rail conditions, railhead profile, crosstie conditions, fastener type, ballast conditions, wheel profile, train braking, track alignment, track surface, lateral and vertical wheel load (static plus dynamic), etc.]. The critical element is rail in long, jointless lengths. The entire system must sustain longitudinal forces in the rail as temperature fluctuates. For years, track maintenance engineers have struggled to control CWR, that is, to lay CWR without building in future problems and to maintain CWR to avoid problems. Most engineers do not believe that this contest has been won; however, knowledge has increased in this area. The intent of this paper is to share Union Pacific's experience and provide help to others who are confronted with similar problems. Most instructions issued by Union Pacific to engineering forces, dispatchers, and train crews are included in this paper.

Union Pacific Railroad is the second largest railroad in the United States, with nearly 23,000 route mi linking western and Gulf Coast ports with the Midwest. Major categories of freight hauled by the railroad are coal, grain, chemicals, automotive parts and machinery, forest products, and intermodal traffic. In 1988 coal was the largest commodity in terms of total revenue ton-miles (28.2 percent), whereas chemicals traffic produced the highest percentage of freight revenue (21.7 percent).

BACKGROUND

The Union Pacific Railroad as it exists today is a combination of the former Union Pacific (UP) and Missouri Pacific (MP), Western Pacific, and the Missouri, Kansas–Texas Railway. The MP began installing continuous welded rail (CWR) on main line tracks in 1955. UP did not begin using CWR on its main tracks until 1969, with complete utilization on curves in 1982. This was partially dependent on the ability to reliably weld premium rail of various metallurgies used in curves. In retrospect, the unknown contributed to the slow integration of CWR into UP railroad operations. Most tract engineers now agree that, for a variety of reasons, CWR has helped more than any other development to reduce the total cost and improve the reliability of the track structure. A long-term goal of Union Pacific is to eliminate every joint in main line trackage, particularly around special track work.

Methods and Research, Union Pacific Railroad, 1416 Dodge St., Room 1000, Omaha, Nebr. 68179.

In the 1960s the process of learning to live with CWR at Union Pacific commenced in earnest, with each year bringing more practical experience. With growing experience on Union Pacific, the rate at which CWR was installed in track increased. This trend was a mixed blessing. Figure 1, information provided by FRA, demonstrates that the number of derailments caused by buckled track for all U.S. railroads increased, year by year, right along with CWR installation, at least through 1980. Derailments declined through 1986, followed by a bottoming out. It appears that during early periods, when there was an increase in the installation of CWR, there was no effective lateral stability of the CWR track system.

This trend was evident on the Union Pacific. Ten years ago, a Union Pacific construction gang under the direction of a young supervisor was installing switch ties on the main line in Nebraska at the location of a future crossover. In the late afternoon, a van train traveling 70 mph hit the weakened track structure and derailed. The supervisor failed to properly de-stress the rail before beginning construction and had a slow order protecting the track only during assigned working hours. Years later, at another location, a section foreman replaced three defective ties under a joint in the morning. He then surfaced the track by hand, but failed to place the appropriate slow orders. Late that afternoon, the track buckled under an eastbound train. In addition to the improper slow orders, the track did not have a full ballast crib or proper shoulder ballast. In 1988, at a third location, a loaded eastbound coal train was traveling 50 mph on a hot afternoon. The train was entering a curve with tight rail. The engineer in control of the train applied the brakes to reduce speed. The track alignment was such that the curve was followed by a short stretch of tangent track followed by a large rigid structure, a bridge. The track buckled between the locomotives and the bridge. The suspect track should have been patrolled earlier in the day because of the hot weather.

Fortunately, no one was seriously injured in these derailments, but they did cause considerable freight, track, and equipment damage, and numerous train delays. In each of these examples, several events or interactions, some understood and some not, contributed to the derailments.

Beginning in the early 1970s, industry, through the Association of American Railroads, and FRA initiated a joint research effort designed to answer many of the questions about how the elements of the track system responded to realistic loads. This program, which is still in existence, devotes resources in time, effort, and capital to study CWR. It was from this base that the technical understanding of the behavior of CWR emerged to complement the accumulated

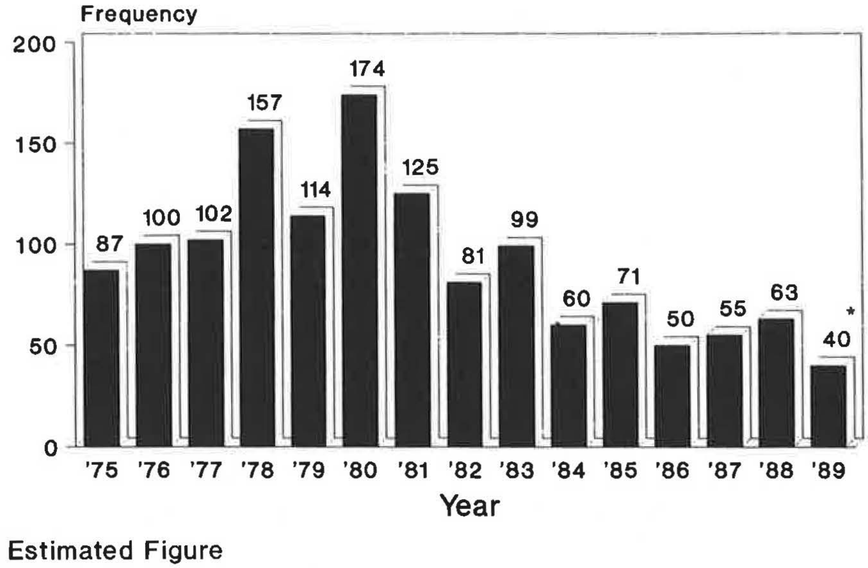


FIGURE 1 Frequency of derailments caused by buckled track, as reported to FRA.

body of practical experience. Analysis and repeated field tests produced data that allowed track maintenance engineers to deal confidently with such concepts as

- The effect of track disturbance on reducing the lateral stability of CWR track and what is required in terms of traffic (load, vibration, and time) to effectively reconsolidate the disturbed ballast while restoring sufficient lateral stability to permit safe operation at regular speeds.
- Track-train dynamics, particularly the notion of progressive bending wave–rail action under vertical wheel loads that provides a partial explanation for lateral track shift under a moving train.
- The idea that the change with traffic and time of rail neutral temperature (zero longitudinal rail stress) is usually toward a lower number. It has been shown in repeated experiments that there is a definite range in which the temperature of rail steel must rise before lateral instability develops. To the extent that the stress-free rail temperature drifts downward, this range is narrowed, increasing the vulnerability of CWR track to lateral buckling.

Implicit in each concept is a reemphasis of the traditional qualities of a good track structure, including adequate amounts of dense ballast, secure rail fastenings, crossties capable of proper distribution of applied loads, and train operating procedures (usually braking) that minimize longitudinal load input at the wheel-rail interface, which are all predicated on the proper installation of the rail. With the change from bolted to CWR track, different quality deficiencies become critical, and the level of risk associated with a potential derailment changes accordingly.

For several years Union Pacific track engineers have been exposed to a "code of conduct" that must be followed if a railroad is to use CWR. The balance of this paper will describe how Union Pacific Railroad translated its own body of experience, tempered with selective reliance on investigative results, into the guidelines that enable the railroad to contend successfully with CWR.

The various railroads that now make up the Union Pacific began track buckling prevention programs in the 1960s with the development of various written instructions regarding rail laying temperatures and handling of track disturbed by maintenance. Eventually, the instructions were developed into formal chief engineer's instructions. The amount of CWR installed in track continued to increase, as did derailments due to buckled track. The following factors contributed to Union Pacific's decision to develop a formal track buckling prevention program:

- Some foremen and supervisors were not comfortable with the use of CWR and did not understand how to manage or work with it.
- Work by the Transportation Systems Center funded by the U.S. Department of Transportation greatly enhanced knowledge about CWR.
- CSX Transportation, then the Chessie Railroad, had developed an effective track buckling prevention training program that included a video and resulted in significant reductions in buckled track on their railroad.
- Union Pacific's top engineering department managers reviewed and understood the benefits of the Chessie program and decided to develop a Union Pacific program.

After the creation of the training program and instructions, plans were made to formally train all track department employees and other engineering employees within the company who could have an impact on buckled track or its prevention. This training is repeated annually, usually in the late winter. The commitment to the program is genuine throughout the organization. The training has been updated periodically since its initial development to include the latest research or information about the prevention of track buckling.

UNION PACIFIC TRACK BUCKLING PREVENTION PROGRAM

The following description is from a Union Pacific Chief Engineer's (CE) Instruction Bulletin (*1*) on track buckling. The bulletin is based on the research and experience previously outlined.

The importance of this information is that track buckling or sun kinks are not acts of God that cannot be controlled. Track buckling is an extraordinary circumstance that can and must be prevented. Compliance with the nine preventive measures discussed next will ensure that sun kinks and pull-aparts are eliminated. The immense benefits to the safety and efficiency of Union Pacific's operation are certainly worth the extra care taken in the prevention of track buckling.

When and Where Does a Track Buckle?

1. The vast majority of all sun kinks occur on hot, sunny afternoons, usually between 2 and 6 p.m. An ambient (air) temperature of 80°F on a calm, sunny day results in a rail temperature of approximately 100°F. Ambient temperatures in excess of 100°F can result in rail temperatures as high as 140°F.

2. Eighty percent of all sun kinks occur in the late spring or early summer. Most occur in April and May, primarily during the season's first hot spell when there are large variations between daytime high and nighttime low temperatures. The problem persists through June and July when the annual peak temperatures are usually first reached.

3. Track buckling is most likely to happen where major track maintenance work, such as tie renewals, undercutting, sledding, plowing, or surfacing and lining, was recently performed. The lateral resistance of track that has been disturbed by one of these means is reduced by more than 50 percent.

4. Sun kinks frequently occur at locations at which substandard track work was performed. An incomplete and improper rail anchor pattern or insufficient ballast section can directly result in a reduction in the longitudinal and lateral holding power of the track, and a lowering of the neutral temperature of the rail.

5. The majority of track buckling occurs in an area in which CWR has been laid. Much buckling occurs where rail has been laid or repaired during the late fall, winter, or early spring. Improper temperature control when rail is laid, or addition of rail during repair of service failures, replacement of detector car defects, or pull-aparts during cold weather can greatly lower the neutral temperature of the rail.

6. Buckling is more likely to occur in track with poor surface and alignment than in track with good surface and line. Minor surface and alignment defects, especially corrugated rail, which increases dynamic loading, coupled with low neutral temperatures can lead to progressive buckling, particularly on sharp curves.

7. More buckling occurs on curves than on tangent track, however buckling on tangent track is usually more severe. Curves almost always buckle outward (C shape), whereas tangent track generally buckles in both directions (S shape). Many sun kinks occur on curves that were surfaced and lined during the winter months and inadvertently lined in (shortened), resulting in a lowering of the neutral temperature of the rail.

8. Much track buckling occurs at the bottom of grades in areas of heavy braking adjacent to road crossings, turnouts, platforms, bridges, and spots of cemented ballast where the rail tends to bunch, thus lowering the neutral temperature of the rail. Running rail or tie movement in a loose ballast section at any of these locations dramatically increases the possibility of buckling.

9. Most buckling happens under a train, with a large percentage occurring under the rear half of the consist. Dynamic forces significantly increase buckling potential. This instability is due to the uplift in the track between the front and rear trucks of a car as related to the bending wave character of the rail, the influence of repeated, heavy wheel impacts on the rail under long trains, and the raising of the rail temperatures (by as much as 20°F) caused by friction between the steel wheels and steel rail. Locomotives and heavy trains can also push or pull rail, especially in heavy traction or on grades, which can increase rail compression and shift neutral temperature.

10. Poor train handling contributes to many sun kinks. The braking action of a train changes the longitudinal forces in the track and can cause significant shifts in the neutral temperature of the rail. Slack adjustments in the train can produce extremely high lateral forces on the rail. Improper train handling in areas where track work was recently performed greatly increases the probability of a sun kink, particularly on grades or in curves.

What Must Be Done To Prevent Buckling?

Ballast Section and Rail Anchor Pattern

A standard ballast section and rail anchor pattern must be maintained. The resistance of the track to longitudinal movement is determined by the lower of either the ballast resistance or the anchor resistance. Simply put, a full ballast section minimizes the possibility of creeping ties and a standard anchor pattern reduces the risk of rail movement. A full ballast section is also required to maximize the lateral resistance of the track. Eighty percent of lateral holding power attributable to ballast resistance is concentrated at the bottom and sides of the ties (with full cribs), and the remaining 20 percent is provided on the ends of the ties (12-in. shoulder). Little additional lateral resistance is obtained by increasing the shoulder width beyond 12 in.

Temperature Control

Continuous welded rail (CWR) and jointed rail must be temperature controlled when installed. Jointed rail must be laid with the proper expansion provided between rail ends. CWR must be laid at the temperature prescribed by the chief engineer for the territory involved. The approved minimum installation temperature for CWR is usually 40 to 45°F above the mean annual temperature for the area and varies from 90°F in some cold, mountainous territories to 115°F in some hotter territories.

The neutral temperature of CWR tends to shift downward toward the optimum ambient temperature with time, because of a number of factors that affect the length and stability of the track, such as surfacing and lining, repairing rail defects, derailment and flood reconstruction, switch installations, running rail and track creep, dynamic forces, and the like. Low neutral temperatures result in extremely high compressive forces in the rail in hot weather, dramatically increasing the possibility of track buckling. This situation must be corrected by cutting out rail to increase the neutral temperature. However, it is important to field-weld these cuts to prevent pull-aparts when the rail is in extreme tension during the winter months.

Rail Repairs in Cold Weather

Service-failed rails, detector car defects, pull-aparts, and other rail repairs undertaken on CWR during cold weather must be accomplished without adding rail. That is, the length of the rail installed to repair the defect must not exceed the length of the rail removed from the track. Sufficient anchors must be removed in both directions from the side of the tie away from the joint(s) to allow rail movement toward the joint(s) only, and then the gap at the joint(s) must be closed using rail expanders, rail heaters, or oil-soaked fiberglass rope placed along the base of the rail and ignited. Once the gap is closed, the rail must be box anchored (every tie) at least 195 ft in each direction from all joints. Standard or compromise joints must be field-welded and insulated joints glued as soon as possible to eliminate the possibility of a pull-apart. The length of the field weld, 1 or 2 in. per weld, must be cut from the rail in track or subtracted from the length of rail to be installed to ensure that rail is not added.

When it is impossible to make repairs as outlined above and rail must be added, the location must be recorded and reported to the track maintenance manager, who must monitor the location as the weather warms in the spring and, at the first sign of any tight rail conditions, must arrange to cut out at least the amount of rail that was added.

Rail in Curves

CWR curves must not be lined in (shortened) unless rail is cut out to compensate for the reduction in neutral temperature. Lining a curve in (i.e., toward the low rail) shortens the curve, resulting in a lower neutral temperature and higher compressive forces in the rail during hot weather. This is particularly critical when surfacing and lining a curve during cold weather with a production tamper or lining with a production liner, since the tamper/liner automatically smooth-lines the curve in when the rail is in tension.

Curves lined in the winter months must be recorded by the track maintenance manager and monitored as warm weather sets in, to ensure that tight rail conditions do not develop. Tight rail in curves can be minimized by lining curves to stakes and balancing the throws. Tight rail in curves can be corrected by lining the curve out or by cutting out excess rail. Good surface and line is particularly important in the prevention of progressive buckling in sharp CWR curves because poor surface and minor alignment imperfections, coupled with low neutral temperatures, can initiate growth to critical levels. Badly corrugated rail in curves also contributes to the problem of progressive buckling and must be corrected by out-of-face grinding or relay of the rail.

Slow Orders

Undercutting, sledding, plowing, surfacing, lining, tie installation, track construction, track rehabilitation or restoration, and any other type of track work undertaken in hot weather that disturbs the roadbed or ballast section must be protected with an appropriate slow order until the ballast section has consolidated under traffic. Consolidation under at least 125,000 gross tons of train traffic is required to restore roughly 50 percent of the lateral resistance lost during surfacing and lining operations (i.e., restoration to 75 percent of original strength). Passage of at least 1 million gross tons (1.0 MGT) is required to re-establish almost all of the original holding power of the track. The chief engineer's instructions spell out the minimum requirements for slow ordering track when track work is performed during hot weather. However, more restrictive measures, such as slower speeds, longer order limits, or longer time limits must be taken when conditions such as a substandard ballast section, insufficient anchor pattern, incomplete spiking, heavy grades, sharper curvature, or proximity to fixed facilities (e.g., bridges, switches, platforms, or road crossings) warrant additional protection. Ordinary or spot track maintenance work that disturbs the track structure should be avoided on CWR during hot weather, to the extent practical.

Inspection

Main lines and sidings must be inspected frequently during hot weather, primarily to detect tight rail conditions in order to take corrective action before the track buckles. Inspection is particularly critical during the first hot spells in the spring (80°F plus) and during extremely hot weather (90°F plus) thereafter. Inspection for tight rail and sun kink locations is most effective between noon and 7 p.m. on hot, sunny days. The inspector should look for extremely kinked or "nervous" rail that is riding up or out of the tie plates or is crowding the shoulder of the plates. The inspector should also look for clusters of high spikes or bad ties, tie movement in the ballast as evidenced by bunching or lack of the ballast at the end of the ties, and running rail as evidenced by anchors not tight against the tie or by shiny marks on the base of the rail where the rail has slipped through the anchors or spikes. An appropriate slow order must be placed until the condition is corrected.

Known tight rail locations can be cut with a saw in the morning while the rail is still cool and in tension. However, extremely tight rail discovered in the heat of the day requiring immediate corrective action will have to be cut with a torch. (Rail in extreme compression will pinch the saw blade, thus precluding the use of a saw for the initial cut.) Rail cut with a torch must then be recut with a rail saw at least 3 in. from the torch-cut ends to eliminate brittle martensite from the ends of the rail. Therefore, at least 6 in. of excess rail must be removed from the track or a replacement rail (15-ft minimum length) must be cut in if removal of less than 6 in. is desired. All cuts must be field welded as soon as practical to prevent rail-end batter and to preclude the possibility of pull-aparts in the winter. The important thing to remember concerning tight rail is "If in doubt, cut rail out."

Temperature Restrictions

Preventive blanket speed restrictions must be applied during extremely hot weather. When ambient temperature reaches or exceeds temperatures shown in Table 1, all trains are restricted as shown in Table 2. In the spring or early summer when the ambient temperature first reaches a daily peak temperature 5° below the temperatures shown in Table 1, the restrictions presented in Table 2 apply. The blanket heat orders will continue to be applied at the 5° lower level for five consecutive days, after which the effective temperature of the blanket heat order may be raised to the maximum level shown in Table 1.

Track maintenance managers and track inspectors must inspect their main tracks via Hy-rail or automobile during the heat of the day when the blanket heat orders are in effect, looking primarily for tight rail and substandard track conditions. The blanket speed restriction guidelines outlined in Instruction Bulletin (*1*) are minimum requirements, and more restrictive measures must be taken when conditions warrant them. Specific tight rail locations must be restricted to as slow a speed as necessary to prevent track buckling and derailments until the rail can be destressed by being cut to relieve the high compressive forces.

Track maintenance managers are responsible for placing blanket speed restrictions on their respective territories. When instructing train dispatchers to issue track bulletins because of extremely hot weather, track maintenance managers must advise the train dispatcher whether the Level 1 or Level 2 heat restriction applies, and the time and location where the track bulletin is to be in effect. There are two general types of time limits for placing the heat restriction in effect:

1. On a day when it is anticipated that the ambient temperature will reach the previously indicated threshold levels, the train dispatcher should be notified in advance (usually in the morning or the night before) that the restriction is to take effect.
2. On a day when it is anticipated that the ambient temperature will not reach the aforementioned threshold levels,

TABLE 1 TEMPERATURE TABLE FOR BLANKET SPEED RESTRICTIONS

STATE	SUBDIVISION/BRANCH	STATIONS	TEMP
CALIFORNIA	LOS ANGELES	YERMO-DAGGETT	105°
		RIVERSIDE-LOS ANGELES	100°
	CIMA	BORAX-YERMO	105°
	OAKLAND	ENTIRE SUBDIVISION	95°
	CANYON	STOCKTON-JAMES JAMES-PORTOLA	95° 90°
	WINNEMUCCA	ENTIRE SUBDIVISION	90°
	BIEBER	ENTIRE BRANCH	90°
NEVADA	ENTIRE STATE		90°
EXCEPT	CALIENTE	CRESTLINE-ISLEN ISLEN-LEITH LEITH-LAS VEGAS	95° 100° 105°
	CIMA	LAS VEGAS — BORAX	105°
UTAH	ENTIRE STATE		90°
EXCEPT	CALIENTE	UVADA-MILFORD	95°
IDAHO	ENTIRE STATE		90°
OREGON	ENTIRE STATE		90°
WASHINGTON	ENTIRE STATE		90°
WYOMING	ENTIRE STATE		90°
NEBRASKA	ENTIRE STATE		90°
COLORADO	ENTIRE STATE		90°
EXCEPT	HOISINGTON	ENTIRE SUBDIVISION	95°
KANSAS	ENTIRE STATE		90°
EXCEPT	COUNCIL GROVE	ENTIRE SUBDIVISION	95°
	HOISINGTON	ENTIRE SUBDIVISION	95°
	WICHITA	ENTIRE SUBDIVISION	95°
	MCPHERSON	ENTIRE BRANCH	95°
ILLINOIS	ENTIRE STATE		95°
EXCEPT	CHICAGO	SALEM NORTH	90°
	PANA	ENTIRE SUBDIVISION	90°
MISSOURI	ENTIRE STATE		95°
OKLAHOMA	ENTIRE STATE		100°
ARKANSAS	ENTIRE STATE		100°
LOUISIANA	ENTIRE STATE		100°
TEXAS	ENTIRE STATE		100°
NEW MEXICO	ENTIRE STATE		100°
TENNESSEE	ENTIRE STATE		100°

Covering main lines and branch lines with maximum operating speeds more than 40 mph.

TABLE 2 SPEED RESTRICTIONS APPLIED DURING HOT WEATHER

Type of Train	Speed Restriction (mph)
Level 1 Heat Restriction[a]	
Passenger trains, light engines, trains with symbol Z that are 5,000 tons or less, and unit double stack trains that are 5,000 tons or less.	None
Trains with symbol Z more than 5,000 tons and unit double stack trains more than 5,000 tons.	60
All other trains averaging less than 90 tons per car or platform.	50
All other trains averaging 90 tons or more per car or platform.	40
Level 2 Heat Restriction[b]	
Freight trains averaging 90 tons or more per car or platform.	40
All other trains (including light engines).	50

NOTE: The Level 1 and 2 heat restrictions may be found in Union Pacific Railroad Timetable No. 7, Special Instructions, Oct. 29, 1989, p. 120.
[a]To be used when ambient temperature is up to 10°F above the temperature shown in Table 1.
[b]To be used when ambient temperature is 10°F or more above the temperature shown in Table 1.

but the levels are subsequently reached, the track bulletin should be issued to take effect immediately.

Unless unusual conditions exist, both of these general types of time limits should be lifted at 9:01 p.m. without issuance of another track bulletin. The removal time of 9:01 p.m. should not be subsequently shortened unless the temperature drops significantly later in the day, in which case the track bulletin can be cancelled before 9:01 pm. All engineering officers, managers, and supervisors must continually monitor the status of track bulletins placed on their territories to ensure that these instructions are appropriately applied. Heat restrictions may be applied at lower temperatures or lower maximum speeds may be specified if, in the judgment of the track maintenance manager, conditions such as heavy grades, sharp curvature, insufficient anchor pattern, substandard ballast section, tight rail, and so forth warrant additional protection to ensure the continued safe operation of trains.

Extent of Speed Restrictions

Speed restrictions placed because of track work must be enforced beyond the limits of the work to ensure that trains have slowed to the desired speed before entering the area of the unstable track. Heavy braking actions and slack adjustments must be made before encountering the newly worked track to minimize track bunching and rail running, and to reduce the dynamic forces created by the movement of the train over the track. Under normal conditions, slow orders should extend at least ¼ mi in each direction from the outside limits of the newly disturbed track. Heavy grades and sharp curves may warrant additional slow-order lengths, particularly where substandard track conditions are present.

Reporting

If a sun kink does occur, it must be reported on the standard form even if the buckling did not result in a derailment. Proper reporting is essential in order to identify problems and trends; thus, it is important that all the information required be completely and accurately reported. This information is then used in developing necessary preventive programs.

UNION PACIFIC RAILROAD PROCEDURES

The following are additional instructions provided to Union Pacific dispatchers, locomotive engineers, and others to help in understanding how the information loop is closed within the Union Pacific Operating Department.

Office Bulletin OB-04-28-89TT 6 STS

Office Bulletin OB-04-28-89TT 6 STS, issued to all dispatchers and officer personnel, reads as follows (*2*):

> Air Brake and Train Handling Rule 1104(C) is now in effect. This rule was written to help prevent derailments caused by track buckling. Rule 1104(C) is intended to be imposed at locations where Engineering forces have performed work disturbing the track structure in such a manner that the risk of track buckling is increased. This rule was designed to be implemented at the discretion of the Manager Track Maintenance (MTM) or supervisor in charge of work and used in conjunction with the speed restriction placed on the disturbed track. Before calling in a track restriction to the dispatcher's office, the MTM or supervisor will evaluate the type of work performed, temperatures expected, and other conditions to determine if implementation of Rule 1104(C) is necessary for that location.
>
> When requested by MTM or Supervisor in charge of work to issue track bulletin due to extreme heat, train dispatcher will be furnished necessary level of heat restriction (Level 1 or 2), time limits, and location limits the track bulletin is to be in effect. Train dispatcher will then make track bulletin as shown below, adding necessary information, and issue to all trains affected:
>
> Level (1 or 2)
> HEAT RESTRICTION APPLIES AS PRESCRIBED BY GENERAL ORDER
> BETWEEN (time) AND (time)
> BETWEEN (Location) AND (Location)
>
> With Rule 1104(C) in effect, slower running time through a speed restriction may be expected.

Air Brake Rule 1104(C)

Air Brake Rule 1104(C), issued to all train crews, reads as follows (*3*):

> Track bulletins or other instructions from proper authority may be issued stating that engineers handle their train in accordance with Air Brake Rule 1104(C) between the stated limits. When proceeding through the limits of the track bulletin or where so instructed, the engineer must handle the train so that track and structures within those limits are subjected to a minimum of train handling generated forces.
>
> Adverse forces are imparted to track and structures as a result of excessive speed, harsh slack adjustment, moderate

to high draft or buff forces, and heavy train braking. These forces are minimized when the engineer uses throttle modulation or low dynamic brake amperage, makes no slack adjustments, and uses no automatic brake while controlling speed through the restriction. To the extent practicable, the engineer will use train handling techniques that reduce adverse forces by making power and brake adjustments prior to or following the restriction, and by minimizing buff or draft forces while carefully controlling speed as the train is passing through the restriction.

Instructions for Locomotive Engineers

Instructions to all Locomotive Engineers reads as follows (*4*):

> The air brake and train handling Rule 1104(C) is designed to prevent Track Buckling (Sun Kinks) from occurring ahead of or beneath your train. When conditions merit, this rule will be used in conjunction with track bulletins that have been issued where engineering is or has been working on the track. It will most frequently be applied at locations where high rail temperatures occur.
>
> Where a temporary speed restriction is set by track bulletin, it is the maximum speed trains are allowed over the limits of the restriction. This does not mean that you are expected to maintain that speed no matter what. There is, in fact, no minimum speed through this type of order. There is nothing wrong with proceeding through a slow order well under the speed limit. There may be times where the best way to reduce train generated forces is to proceed very slowly. There may even be times when you will need to allow the train to come to a full stop, such as avoiding the excessive force buildup which can occur when making running releases. There are numerous other methods for reducing in-train forces, allowing speed to drift up by entering the restriction at low speed, allowing speed to drift down after entering at the speed of the restriction, using an air brake/dynamic brake balance in place of dynamic brakes only, or even using a little power against air to maintain a uniform speed and force. Heavy braking in or approaching disturbed track must always be avoided because the braking forces are trying to push the rail ahead and may cause disturbed track to buckle. Heavy dynamic or engine braking must also be avoided for the same reason. Remember that light forces spread evenly throughout the train are much better than a heavy and concentrated force.
>
> When ascending a grade through a restriction, the most desirable technique would be to enter the restriction at the speed you are allowed. As the engines pass over the disturbed track, gradually reduce the power, allowing speed to reduce slightly, but not so much that your train will stall before it clears the restriction. As the locomotive clears the restriction, you may gradually increase the throttle to bring your speed back up to that allowed until the remainder of your train clears the restriction. This method allows energy stored in the train to partially maintain your momentum while reducing the head-end draft forces. This is also a good method to use on level track when the restriction is in a curve. Take care not to shut off so much or so fast that the train stalls or slack runs in. When using this procedure on level track, prepare your train well in advance so that all train brakes are fully released and the power is uniform throughout the train. When practicable, avoid making power changes while the locomotive is in the restriction.
>
> On a slight descending grade where the automatic air is not needed and the dynamic brake is in use, enter the restriction at a slower speed than the restriction allows. Then, gradually reduce the dynamic brake, allowing a slight acceleration, but not enough to allow the speed to go above authorized speed. As the locomotive clears the limits, increase your dynamic enough to prevent the speed from going above that authorized. On heavy descending grade, use a balance of dynamic brake and train brakes. Make minimum or split service reductions sufficiently ahead of the restriction to ensure propagation of braking has ceased before the head-end enters the restriction. If speed drops, gradually reduce the dynamic to allow the train to continue to roll. If the air brakes and dynamic is too much retarding force, gradually reduce the dynamic. If necesssary, ease the dynamic off completely and work light power to maintain your speed. These operating practices, as well as others you may commonly use, will allow you to comply with the intent of Rule 1104(C). The professional locomotive engineer has a considerable repertoire of train handling techniques.
>
> Avoiding track buckles by reducing the forces transmitted to the track is in the hands of the locomotive engineer. Control of the power, automatic brake and the dynamic brake in compliance with Rule 1104(C) is the best way of avoiding a track buckle. If any of the operating practices discussed here are not familiar to you or if you have other questions regarding the intent or application of this rule, you should contact your Manager of Operating Practices. With your help, track buckling derailments can be eliminated.

CONCLUSION

From 1988 to 1989 the Union Pacific reduced total derailment costs by approximately $25 million. A major factor in this reduction was the improvements made in the area of reduced track buckling derailments.

Year	*No. of Buckling Occurrences*	*Total Derailment Costs ($) Plus Additives*
1988	11	3,731,000.00
1989	3	70,000.00

These statistics emphasize the benefits of having an effective and vigorously enforced track buckling prevention program in place.

REFERENCES

1. *Instruction Bulletin CE-88-006-T.* Union Pacific Railroad, Omaha, Nebr., 1988.
2. *Office Bulletin OB-04-28-89TT 6 STS.* Union Pacific Railroad, Omaha, Nebr., 1989.
3. *Air Brake Rule 1104(C): Train Handling Over Disturbed Track.* Union Pacific Railroad, Omaha, Nebr., n.d.
4. *Instructions to all Locomotive Engineers.* Union Pacific Railroad, Omaha, Nebr., April 28, 1989.

Effectiveness of Various Schemes in Controlling the Behavior of Continuous Welded Rail

MAX A. FERGUSON

The effectiveness of the various schemes railroads have used to control the behavior of continuous welded rail (CWR) over the past 15 years is discussed in this paper. Considerations and procedures used in the investigation of train derailments in which track buckling may be a causal factor are addressed, and 16 derailments are reviewed in detail. A pattern of several factors was found that either lowered the neutral rail temperature or materially reduced the lateral stability of the track. These factors were longitudinal rail creep, the chording inward of curves, addition of too much rail, and failure to sufficiently consolidate the ballast after it has been disturbed before trains pass at scheduled speeds. Rail creep may be reduced by adding more rail anchors or reducing train speeds and braking forces until the ballast has been compacted by trains or by mechanical methods. Reference staking under certain conditions will determine if curve chording has taken place and if adjustments will be necessary. After CWR is cut in cool weather, rail adjustments need to be made in order to avoid the addition of rail. After track is disturbed at high temperatures, the ballast must be adequately consolidated before trains are allowed to resume higher speeds. Railroads must have clear instructions on maintenance practices that could result in track buckling and train personnel to understand the application of these instructions.

Under the provisions of the Accident Reports Act (Title 45, U.S. Code), FRA has the authority to investigate train accidents. FRA's Office of Safety initiates investigations of serious railroad accidents and assigns members of its field force to the task of gathering factual information, determining a probable cause, and preparing a report. These reports are then submitted to the Washington, D.C., office for review and final approval. Information regarding each accident is published annually in the *Summary of Accidents Investigated by FRA*.

For the past 17 years, the author, as a regional track engineer in the southeastern United States involved with the enforcement of FRA's Track Safety Standards, has participated in many of these railroad accident investigations, particularly those in which track conditions may have been a causal factor. Special attention has been given derailments that may have been caused by insufficient lateral track stability, commonly called buckled track in the railroad industry.

Many of the potential ingredients for track buckling in continuous welded rail (CWR) are known, such as high rail temperature, poor maintenance practices during previous track work, train and dynamic braking on descending grades and in curves, and so forth.

A less-publicized consideration faced by the derailment investigator when considering the possibility of track buckling under a train after the lead locomotive has passed over the point of derailment is how and why the wheels of the first car or cars in the derailed train left the rail. Another question is why, in several cases, some rail vehicles negotiated the track at the point of derailment whereas other cars derailed. These questions need to be answered when possible causes are considered. The investigator inspects the first cars to derail, determines how the derailed wheels were positioned with respect to the track structure after they came to a stop, and notes all the wheel and flange marks at the scene. The investigator then may ask, if it is assumed that the track buckled under the train, "Would it be possible for the wheels of this loaded or empty car to derail in this manner?"

For example, one pattern noted in several derailments on curves, in which other evidence substantiated buckled track, was that loaded cars traveling in an unstable equilibrium on strong CWR track derailed to the low side, or where the wheel or wheels cross over the inner rail of a curve. Often one car derails, one or more negotiate the buckling, another derails, and so forth. In a curve the track buckles to the outside of the curve, but in the example just given the wheels derail in the short reverse curve preceding and made by the buckled-out portion of track (see Point *A*, Figure 1). The inside rail of the original curve becomes the outside rail of that small reverse curve. With a loaded car in an unstable equilibrium, the weight on the wheels on the inner rail of the curve is significantly lower than that on the outside rail (see Figure 1). About the only other situation causing a car to derail to the inside of a curve involves a train experiencing excessive draft forces resulting in a stringlining effect. Empty cars may derail because their wheels cross over either rail. On track with a weak tie condition, the wheels of loaded cars may turn either rail outward far enough to cause the cars to drop inside the track gage, or they may spread the track and all following cars will probably derail.

Given the information that a buckling could have occurred, on the basis of the presence of previously mentioned factors and the manner in which the cars derailed, it is then basic to the accident investigation to determine the maintenance history of the portion of track involved. At this point, the investigator must also determine the railroad's maintenance instructions for laying and maintaining CWR. If all or most of the facts concerning the maintenance history can be devel-

Federal Railroad Administration, Suite 440, North Tower, 1720 Peachtree Road, Atlanta, Ga. 30309.

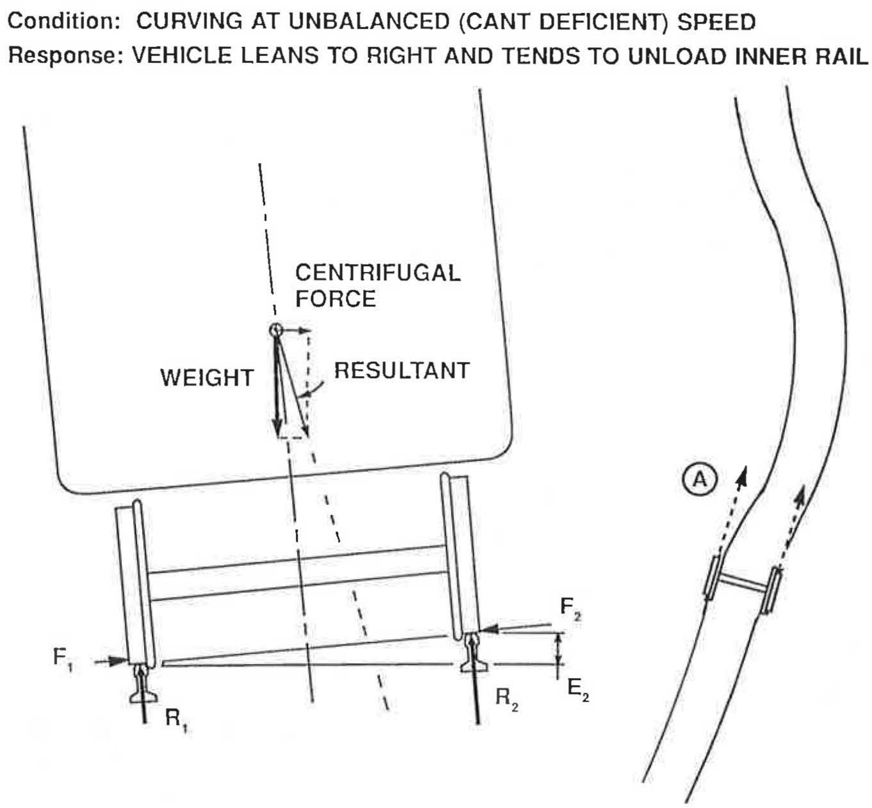


FIGURE 1 Rail vehicle negotiating a buckling in an unstable equilibrium.

oped, the following can then be determined: (a) Was the stage set for buckling by an improper maintenance practice, given a high rail temperature and forces induced by the train? (b) Were the railroad's instructions followed? (c) Did the railroad's instructions include steps to prevent this type of situation, and if so, were they clearly understood by railroad employees?

In cases in which the buckling was not seen by the train crew before the train passed over it, or in which the track in the vicinity of the point of derailment was destroyed during the accident, the probable cause is based on circumstantial evidence. All information must be considered, along with train-induced forces. Information from interviews with the train and engine crews, and from speed and event recorders, if available, must be used to determine the speed, how the train was handled, and what in-train forces may have developed. Determination of these forces is best accomplished by using a train dynamics analyzer or simulator. Given train handling scenarios, the train consist, car information, tonnage, and the track profile and curvature information, the draft or buff forces may be approximated for the car that derailed first at the point of derailment. If these forces are within a reasonable range, it may be concluded that lateral track displacement was not caused by in-train forces, but that these forces contributed to an incipient thermal buckling. To support a probable cause of track buckling that occurs under a train, the question "Why did it buckle?" must be answered.

So it is at this point that the subject of this paper, the effectiveness of various schemes in controlling the behavior of CWR, is addressed. The effectiveness may best be determined by reviewing a number of derailments in which something evidently went wrong and the track buckled. It will be determined whether existing instructions were clearly understood and followed, whether existing instructions correctly address the subject, and whether more instructions are needed on some or all railroads.

A review of the circumstances involved in 16 derailments with a probable cause of buckled track is outlined in Tables 1 and 2. In the 15-year period covered by the data in these tables, a pattern of what went wrong and how railroads changed their instructions to counter the problems can be seen. It is clear that in all cases track maintenance took place from 1 hr to 7 months before the derailment. This work resulted in too much rail in the track or, in other words, a significant reduction in neutral temperature, to below that desired to prevent buckling caused by relatively high temperatures and train-induced forces. Neutral temperature is defined as the rail temperature at which anchored CWR is free of longitudinal stress, that is, not in tension or compression. Railroads endeavor to install or adjust CWR to an optimum neutral temperature for the geographical area, so that it will withstand the extreme heat and cold. In Georgia, for instance, the desired neutral rail temperature is between 90 and 100°F.

Ten of the 16 derailments involved descending grades where the train was braking at the time of derailment or where previous trains had braked. Dynamic, independent, and automatic train braking all cause significant longitudinal rail movement on track that has been recently disturbed by maintenance. This movement occurs even on track with the usual number of rail anchors. When the ballast is disturbed, it does

TABLE 1 ACCIDENT SUMMARY

Derailment No.	Descending Grade	Recent Work	Rail Creep	Curve or Tangent	Chording a Factor	Slow Order: Not Placed	Slow Order: Not on Long Enough
1	Yes	Yes	Yes?	C	?	X	
2	Yes	Yes	Yes	T	No		X
3	No	Yes	Yes	T	No		
4	No	Yes	No	C	Yes		
5	Yes	Yes	Yes	T	No		
6	Yes	Yes	No	C	Yes		
7	Yes	Yes	No	C	Yes		
8	Yes	Yes	No	C	Yes		
9	Yes	Yes	No	C	No		
10	Yes	Yes	No	C	Yes		
11	No	Yes	Yes	T	No		
12	No	Yes	No	T	No		
13	Yes	Yes	Yes	T	No	X	
14	Yes	Yes	No	C	No	X	
15	No	Yes	No	C	Yes		
16	No	Yes	No	C	Yes		
Total	10 yes	16 yes	6 yes		7 yes		

Blanks indicate either slow order was placed, or since considerable time had passed sinced maintenance work a slow order did not remain in effect.

not have enough resistance to overcome the longitudinal force transmitted to it by the ties. In this paper, such movement will be called longitudinal creep. At places where creep is impeded, compressive stress builds up in the CWR, or tensile stress is decreased if the rail is in tension. These places include turnouts, vertical curves at the bottom of grades, horizontal curves, and bridge approaches. The neutral temperature at those locations is reduced to below the desirable temperature. At high temperatures thermally induced, static, longitudinal compressive forces build up, and a train traversing the location contributes sufficient dynamic forces, both longitudinal and lateral, to cause buckling, the amplitude of which is increased with the passage of the cars in the train. In 4 of the 10 derailments on descending grades, it appears that rail creep was a significant factor in causing the buckling. It was also a factor in two derailments on level track.

On the basis of this experience, it appears that railroads have a problem in adequately controlling creep even though rail anchors are applied to their respective standards. What controls do railroads have? Longitudinal rail creep may be reduced by slowing train speeds and reducing braking forces until the ballast has been compacted by several tonnage trains. Some railroads use a machine method of ballast consolidation, such as dynamic stabilizers or compactors, to simulate track vibration induced by train movements and reduce the necessity for slow orders. The application of additional rail anchors in areas where heavy braking is expected aids in reducing creep. In the six cases discussed in which rail creep was a factor, the controls failed for several reasons. In two cases, a slow order was never placed; in one case it evidently was not in force long enough. In the other cases it is not known if or how long orders were in force. In the two cases in which an order was not placed, the carrier had slow order requirements, but they were not clearly understood by the people performing the work. Rail creep has been and remains a major problem. All railroads need to review their instructions to see that creep is properly addressed, particularly at those critical locations mentioned previously.

In 7 of the 10 derailments that occurred in curves, one of two conditions, or a combination of both, evidently existed:

1. After track on a curve was disturbed by maintenance that reduced lateral and longitudinal stability, the curve shifted inward (chorded in) during cold weather before the ballast section was restored or was sufficiently compacted by train movements, and the track stayed in this position until the time of derailment.

2. During a surfacing and alinement procedure at cool temperature, the curve was thrown inward more than outward. In one derailment investigation this was documented through comparison of string line notes before and after the curve was lined. This phenomenon may also be determined by comparing track geometry car information before and after alinement work.

TABLE 2 REVIEW OF CIRCUMSTANCES OF DERAILMENTS

DATE, TIME AND AMBIENT TEMPERATURE IN F°	ALINEMENT AT POINT OF DERAILMENT RAIL, GRADE (D-DESCEND, A-ASCEND)	TRAIN HANDLING METHOD AND SPEED	FIRST CARS IN TRAIN TO DERAIL	TRACK MAINTENANCE HISTORY	RAILROAD INSTRUCTIONS	REASON FOR BUCKLING
(1) MAY 1974 1:05 P.M. 86°	3° CURVE 115 CWR 0.96% D	THROTTLE, NO BRAKING. 58 MPH	1ST PASSENGER CAR PLUS 10 FOLLOWING	THE DAY OF THE ACCIDENT 14 TIES REPLACED IN 50 FT., AND TRACK SURFACED.	SLOW ORDER TO BE PLACED WHEN TIMBERING AND SURFACING.	CWR DISTURBED HIGH TEMPERATURE AND NOT PROTECTED BY SLOW ORDER. NO WAY OF KNOWING NEUTRAL TEMPERATURE.
(2) JULY 1980 3:35 P.M. 99°	TANGENT 132 CWR. AT LEAVING END OF BRIDGE 0.3% D	53 MPH	72ND, MTY, PLUS 39 FOLLOWING CARS	FIVE DAYS BEFORE ACCIDENT THE TRACK OFF THE END OF 60-FOOT OPEN DECK THRU PLATE GRINDER (NOT ANCHORED) WITH 156 FT. BALLAST DECK APPROACH WAS SPOT SURFACED. TRACK TIED AND SURFACED IN FEB. 1980 AT 26°.	SLOW ORDER TO BE PLACED WHEN SPOT SURFACING ABOVE 85°. THIS WAS DONE, BUT ORDER LIFTED BEFORE DERAILMENT.	LONGITUDINAL CREEP, WHEN SURFACED, LOWERED NEUTRAL TEMPERATURE AT END OF BRIDGE. BUCKLE OCCURRED UNDER TRAIN.
(3) JULY 1980 6:55 P.M. 93°	TANGENT 136 CWR. JUST AHEAD OF RAILROAD CROSSING DIAMOND. LEVEL	42 MPH	10TH, MTY, PLUS 18 FOLLOWING	RAIL CROSSING DIAMOND RENEWED IN JANUARY, ALSO SURFACED THEN AT 49°. A 60-FOOT OPEN DECK BRIDGE AHEAD OF AND NEAR DIAMOND. NO ANCHORS ON BRIDGE.	NONE	WHEN THE CWR WAS CUT TO INSTALL CROSSING, RAIL CONTRACTED ACROSS UNANCHORED BRIDGE, AND TOO MUCH RAIL ADDED LOWERING NEUTRAL TEMPERATURE. THERE WAS ONE INCH OF RAIL MOVEMENT ACROSS THE BRIDGE.
(4) JUNE 1982 3:34 P.M. 90°	2° CURVE 132 CWR. LEVEL	NO. 5 THROTTLE, NO BRAKE. 50 MPH	6TH CAR, A LOADED TRI-LEVEL TO LOW SIDE, PLUS FOLLOWING 15 CARS	TRACK SURFACED IN OCT. 1981, WHEN LOW TEMPERATURE REACHED 29°.	NONE	CWR EVIDENTLY CHORDED INWARD DURING OR AFTER SURFACING, REDUCING NEUTRAL TEMPERATURE AND STAYED IN THIS POSITION UNTIL BUCKLED UNDER TRAIN ON A DAY WHEN TEMPERATURE WAS ONE OF THE HIGHEST SINCE OCT.
(5) MARCH 1985 12:30 P.M. 76°	TANGENT 132 CWR. JUST AHEAD OF FACING POINT TURNOUT 0.7% D	NO. 7 DYNAMIC BRAKE. 46 MPH	77TH, MTY, 80TH, LOAD, THRU 100TH.	NEW TURNOUT INSTALLED IN DEC. 1984. WHEN SURFACED IN FEBRUARY, LOW TEMPERATURE WAS BETWEEN 28° AND 57°.	NONE	WHEN TURNOUT INSTALLED AND RAIL WAS CUT, IT EVIDENTLY CONTRACTED BECAUSE OF COOL TEMPERATURE, TOO MUCH RAIL MAY HAVE BEEN ADDED. CREEP MAY HAVE ALSO OCCURRED DURING AND AFTER SURFACING. TRACK BUCKLED UNDER TRAIN IN DYNAMIC BRAKING MODE.
(6) JULY 1985 3:42 P.M. 93°	3° CURVE 132 CWR. JUST OFF LEAVING END OF 600-FT. OPEN DECK BRIDGE (ANCHORED). 0.44% D	NO. 6 THROTTLE WITH 12 LB. AUTOMATIC BRAKE PIPE REDUCTION. 43 MPH	28TH CAR, LOAD, TO INSIDE OF CURVE, THRU 60TH CAR.	CURVE ON LEAVING END OF BRIDGE UNDERCUT IN FEB. 1985, WHEN LOW TEMPERATURE REACHED 15°. SURFACED SAME MONTH WITH TEMPERATURE RANGE 25° TO 53°.	NONE	WHEN TRACK WAS UNDERCUT AND SURFACED, APPARENTLY CWR CHORDED INWARD AND STAYED IN THIS POSITION UNTIL BUCKLED UNDER TRAIN IN TRAIN BRAKE MODE AT HIGH TEMPERATURE. CREW SAW SOME MISALINEMENT ON APPROACH TO SCENE.
(7) MARCH 1986 2:50 P.M. 77°	6° 15' CURVE 132 CWR 1% D	NO. 3 DYNAMIC BRAKE. 40 MPH	LEAD WHEELS OF 47TH CAR, LOAD, DERAILED TO INSIDE OF CURVE. TRAILING TRUCK OF 48TH, AND 49TH THRU 83RD DERAILED.	OUTSIDE RAIL OF CURVE HAD BEEN LAID AND HEATED TO 100° IN DEC. 1985. CURVE SURFACED IN FEB. 1986, DURING PERIOD WHEN TEMPERATURE REACHED AS LOW AS 17°.	NONE	CURVE SURFACED DURING COOL WEATHER AND CHORDED INWARD. BALLAST WAS ADDED AND TRACK COMPACTED BY TRAINS IN THIS POSITION. TRACK STAYED AT THIS LOCATION UNTIL IT BUCKLED OUT ON WARM DAY UNDER TRAIN.
(8) MAY 1986 3:40 P.M. 91°	4° 30' CURVE 132 CWR 1% D	NO. 4 DYNAMIC BRAKE. 30 MPH	60TH, MTY, LEAD TRUCK DERAILED TO OUTSIDE, 61 AND 62 STAYED ON, 63RD, LOAD, SPREAD THE TRACK. THE 64TH THRU 76TH DERAILED.	CURVE WAS SURFACED IN FEB. 1986, WHEN TEMPERATURES REACHED AS LOW AS 17°. CURVE NOTES ALSO INDICATED, WHEN ALINEMENT MADE AFTER SURFACING, THAT TRACK WAS LINED IN MORE THAN OUTWARD.	NONE	CURVE SURFACED DURING COOL WEATHER AND CHORDED INWARD. BALLAST WAS ADDED AND TRACK COMPACTED BY TRAINS IN THIS POSITION. STAYED AT THIS LOCATION UNTIL IT BUCKLED UNDER TRAIN. LINING INWARD MAY HAVE CONTRIBUTED.
(9) JUNE 1986 3:47 P.M. 92°	2° CURVE 112 & 115 CWR. 0.77% D	NO. 6 THROTTLE WITH MINIMUM TRAIN BRAKE REDUCTION.	63RD, LOAD, SPREAD THE TRACK. BALANCE THRU 86TH DERAILED.	THE OUTER RAIL OF CURVE WAS LAID IN MARCH WITHOUT HEATING WHEN TEMPERATURE RANGED BETWEEN 35° AND 53°.	CWR TO BE HEATED WHEN LAYING TO A RAIL TEMPERATURE OF 80°.	RAIL INSTALLED AND ANCHORED AT A LOW NEUTRAL TEMPERATURE. TRACK BUCKLED UNDER HEAVY TRAIN.
(10) JUNE 1986 3:10 P.M. 93°	6° CURVE 132 & 136 CWR. 1.2% D	NO. 8 DYNAMIC BRAKE. 35 MPH	53RD, LOAD, TO INSIDE, 63RD, LOAD, TO INSIDE PLUS 54 MORE CARS.	THE OUTER RAIL OF THE CURVE WAS LAID IN DEC. 1985, AND HEATED TO 95°. IN MARCH 1986, ONE INCH OF ELEVATION WAS REMOVED FROM THE CURVE BY SURFACING AT A TEMPERATURE BETWEEN 49° AND 59°. A FEW DAYS LATER, THE LOW REACHED 22°.	CWR TO BE HEATED WHEN LAYING TO A TEMPERATURE OF AT LEAST 80°. NONE ON SURFACING DURING COOL WEATHER.	THE REMOVING OF ELEVATION REDUCED LATERAL RESTRAINT. DURING PERIOD OF COLD WEATHER SHORTLY AFTERWARD CURVE EVIDENTLY CHORDED INWARD AND STAYED IN THIS POSITION UNTIL IT BUCKLED UNDER TRAIN IN HEAVY DYNAMIC.

TABLE 2 (*continued on next page*)

TABLE 2 (*continued*)

DATE, TIME AND AMBIENT TEMPERATURE IN F°	ALINEMENT AT POINT OF DERAILMENT RAIL, GRADE (D-DESCEND, A-ASCEND)	TRAIN HANDLING METHOD AND SPEED	FIRST CARS IN TRAIN TO DERAIL	TRACK MAINTENANCE HISTORY	RAILROAD INSTRUCTIONS	REASON FOR BUCKLING
(11) JULY 1986 4 P.M. 90°	TANGENT AT LEAVING END OF 670-FOOT OPEN DECK TRESTLE. 115 CWR. LEVEL	NO. 8 THROTTLE. 47 MPH	79TH, LOAD, PLUS 11 FOLLOWING. CARS EVIDENTLY SPREAD THE TRACK.	IN JUNE, FOULED BALLAST WAS STRIPPED OUT AND FRESH BALLAST APPLIED FOR 25 FEET AT LEAVING END OF BRIDGE. CWR ON BRIDGE DID NOT HAVE RAIL ANCHORS.	NO SPECIFIC INSTRUCTIONS CONCERNING ANCHORS ON CWR ON BRIDGES OVER 300 FT. SLOW ORDER WAS LEFT ON FOR 24 HOURS AFTER TRACK WORK IN JUNE.	EVIDENCE INDICATED 2 5/8 INCH LONGITUDINAL RAIL MOVEMENT ON TRESTLE. RAIL EXPANDED ON UNANCHORED BRIDGE REDUCING NEUTRAL TEMPERATURE WHERE TRACK WORK TOOK PLACE. TRACK BUCKLED AT THAT POINT UNDER TRAIN.
(12) JULY 1986 4:10 P.M. 98°	TANGENT 132 CWR. LEVEL	NO. 5 THROTTLE. 18 MPH	69TH, LOAD, PLUS NEXT 7 CARS.	CWR WAS LAID IN 1983 AND 1984, TRACK ALINEMENT AND SURFACE WAS IRREGULAR AT TIME. SURFACED AND LINED IN AUG. 1985. IN WINTER PREVIOUS TO DERAILMENT, NUMEROUS SHORT RAIL PLUGS WERE CUT INTO REMOVE DEFECTIVE RAIL & FIELD WELDS MADE AT TEMPERATURES AS LOW AS 24°.	NONE THAT WERE SPECIFIC.	STRAIGHTENING IRREGULAR ALINEMENT AND SURFACE WOULD LOWER NEUTRAL TEMPERATURE. CUTTING CWR AND WELDING IN PLUGS WITHOUT ADJUSTMENT IN COLD WEATHER WOULD ADD TOO MUCH RAIL TO THE TRACK. TRACK BUCKLED UNDER TRAIN.
(13) JULY 1986 4:24 P.M. 90°	TANGENT, 132 CWR AT RECEIVING END OF 164-FOOT OPEN DECK BRIDGE WITH ANCHORS. 0.2% D	NO. 5 THROTTLE. 45 MPH	24TH, LOAD, TURNED RAIL OVER, 25TH, MTY, CROSSED OVER RAIL, 26TH THRU 38TH DERAILED.	CWR SURFACED A FEW HOURS BEFORE THE DERAILMENT WITH RUNOFF MADE TO END OF BRIDGE. NO SLOW ORDER PLACED ON TRACK.	RULE REQUIRES SLOW ORDER AFTER SURFACING OVER 85°, BUT WAS MISUNDERSTOOD BY TRACK WORKERS.	LATERAL RESTRAINT WAS REDUCED BY SURFACING AT HIGH TEMPERATURE WHEN NEUTRAL RAIL TEMPERATURE WAS UNKNOWN. RAIL CREEP BY TRAIN INVOLVED AND PREVIOUS TRAINS HAD LOWERED NEUTRAL TEMPERATURE RESULTING IN BUCKLE UNDER TRAIN.
(14) AUG. 1986 3 P.M. 90°	3° CURVE 100 CWR. 0.7% D	NO. 8 DYNAMIC BRAKE. 32 MPH	92ND, MTY, TO LOW SIDE, 95TH, MTY, ONE TRUCK TO HIGH SIDE, 96TH, MTY, ONE TRUCK TO LOW SIDE, 98TH, LOAD, TO LOW SIDE PLUS 120TH THRU 123RD.	THE DAY PRIOR TO THE DERAILMENT, A TRACK GANG SURFACED THE TRACK AT 91° AND DID NOT PLACE A SLOW ORDER.	CLEAR INSTRUCTIONS WERE NOT AVAILABLE TO FOREMAN IN CHARGE.	TRACK SURFACING REDUCED LATERAL RESTRAINT. TRACK BUCKLED UNDER TRAIN IN DYNAMIC BRAKING MODE. SLOW ORDER WAS NOT PLACED AND THE NEUTRAL RAIL TEMPERATURE WAS UNKNOWN.
(15) APRIL 1987 2:35 P.M. 83°	1° 47' CURVE 122 CWR LEVEL	NO. 8 THROTTLE. 25 MPH, CREW FELT LURCH OVER P.O.D.	20TH, LOAD, CROSSED OVER OUTER RAIL, 22ND, 24TH, 29TH THRU THE 53RD DERAILED.	THREE DAYS PRIOR TO THE DERAILMENT, A TIE GANG INSTALLED TIES AT 50° TO 57°. BALLAST SECTION WAS NOT FULLY RESTORED.	NO REFERENCE STAKES SINCE THE INSTRUCTIONS WERE TO STAKE CURVE OVER 1° IF WORKED UNDER 50°. 25 MPH ORDER PLACED ON TRACK.	CURVE SHIFTED INWARD AFTER DISTURBING DUE TO COOL TEMPERATURE, AND INADEQUATE BALLAST SECTION. TRACK BUCKLED ON WARM DAY UNDER LOCOMOTIVES.
(16) AUG. 1988 2:55 P.M. 95°	6° 50' CURVE. 132 & 136 CWR LEVEL	NO. 6 THROTTLE. 34 MPH CREW SAID THEY SAW BUCKLE.	24TH, LOAD, SPREAD TRACK. 25TH THRU 53RD DERAILED.	FOUR DAYS PRIOR TO THE DERAILMENT, TRACK WAS SURFACED BETWEEN 70° AND 80°. THE DAY PRIOR TO THE DERAILMENT, THE LOW REACHED 56°.	NO REFERENCE STAKES SET SINCE IT WAS OVER 50°. A 25 MPH WAS ON TRACK IN THIS AREA AND TRAIN SHOULD HAVE BEEN COMPLYING.	CURVE SHIFTED INWARD DURING COOL TEMPERATURES AFTER BEING DISTURBED BY SURFACING. BUCKLED BEFORE TRAIN ARRIVED AT 95° THE HIGHEST TEMPERATURE SINCE SURFACING.

Both of the above conditions may reduce the rail neutral temperature to an undesirable level. The shifting due to cold temperatures may sometimes be observed by inspection, but it often is so uniform that it goes unnoticed.

It should be noted that curves may shift inward during cold temperatures, even if the ballast section was not recently disturbed. This has occurred at locations where the shoulder ballast section on the inside of curves is not sufficient to resist the chording effect from tension that developed at extremely cold temperatures, even though the rail may have been at the desired neutral temperature before it moved inward. Curves may also shift if some recent rail maintenance work (in which no ballast was disturbed) caused a change of neutral temperature to a level higher than desirable, for instance, if rail was installed at, or overheated to, a rail temperature of 125°F. When the rail later cools, high tensile forces in the CWR cause the track in the curve to overcome the lateral resistance of even a well-compacted ballast section and shift inward.

The chording phenomenon, caused by high tensile forces in the CWR, could also be aided by a dynamic stringlining effect that results from large draft forces that develop in trains being pulled up a grade while on a relatively sharp curve. A neutral temperature that is too high may also result in the pulling apart of CWR at a joint or its breaking at a stress riser during cold weather.

One railroad in the South recognized the curve-shifting problem in CWR track many years ago and has instructions to compensate for the problem. Before track on a curve is surfaced or otherwise disturbed at or below a rail temperature of 50°F, reference stakes are set at several locations around the curve. The amount of movement at each stake is recorded one week after the curve is surfaced. If there is an average movement of 1 in. or more, the track must be lined out or slow ordered before hot weather.

As stated earlier, an analysis of the 10 of 16 derailments that occurred in curves showed that 7 of the curves evidently chorded inward during or shortly after surfacing during a period of cool weather.

Reference stakes were not set in any of the cases. Two derailments occurred after the railroad issued reference stake instructions. Stakes were not set because on the days of surfacing, the temperature was more than 50°F. In several cases it was noted that the temperature was near 50°F at the time of surfacing, but dropped within a few days after surfacing and before the ballast was adequately dressed or sufficiently compacted by train traffic. It is entirely possible that all seven derailments could have been prevented had the staking procedure been followed. It is therefore concluded that whenever work involving CWR (laying rail, surfacing, undercutting, or installing ties) is performed in curves, a controlled method for measuring lateral track movement must be set up before the work begins, so that any appreciable change in alinement that occurs during the work or before the ballast is properly consolidated can be recorded. Adjustments can then be made before hot weather. Railroads that do not have these controls should consider instituting them. Railroads that have instructions for staking when the temperature is less than 50°F should consider the consequences of a temperature that is more than 50°F on the day of the work and drops in the next few nights before the ballast has been consolidated.

Once the chording phenomenon on curves is understood, another possibility must be considered. When a curve is disturbed and lined at extremely high temperatures, it can be lined to the outside with relative ease. If this is overdone, the neutral temperature may be raised too high, as in the previously mentioned overheating of the rail during installation. If the rail stays hot until the ballast has consolidated, the track on the curve will stay in this position until it turns cold and the tension becomes so great that it overcomes the restraining friction force of the ballast and chords inward, thus possibly lowering the neutral temperature to below that desirable. The greater the degree of curvature, the greater the forces trying to shift the track inward. Again, controls must be in place to monitor this type of situation.

In derailments 1, 13, and 14 the temperature was high when the maintenance work was performed, no slow order was placed, and the accidents occurred at locations susceptible to buckling—two on curves and one on a bridge approach. Some previous event at these locations reduced the neutral temperature to below the desirable level, causing the rail to be under considerable compression in the hot weather at the time of the derailments. The disturbance of the ballast by the recent work reduced the lateral track restraint, and the addition of train-induced forces buckled the track. In those three cases the railroad employees at the scene did not correctly understand the instructions for placing slow orders during hot weather. A slow order either would have prevented the derailments or at least would have reduced the damage caused by the derailment.

When railroad personnel do not know the rail neutral temperature, they do not know if they are disturbing the track above that temperature. Therefore the track must be covered with a slow order after it has been disturbed. Instructions calling for a slow order at temperatures near the desired neutral temperature for the area may not be sufficient. This consideration, along with the possibility of increased longitudinal rail creep with increased speed, raises the question of whether slow orders should be placed, regardless of temperature, after the track has been disturbed and left in place until the ballast has consolidated.

Derailments 3, 5, and 12 involved cutting CWR during periods of low rail temperature. In Derailment 3, a new railroad crossing diamond (frogs) was installed during cool weather several months before the derailment. Evidently, when the CWR was cut to take out the old diamond, the rails contracted, and too much rail was added when the new diamond was installed. Immediately in the approach to the rail crossing was a 60-ft open-deck bridge on which no rail anchors were installed. The rail creep caused by the rail expansion across the bridge and the impeding effect of the diamond resulted in the build-up of compressive stress on the ballasted track, which in turn caused the neutral temperature to be below the desired level. The track buckled under a train at an ambient temperature of 93°F on a short stretch of ballasted track between the bridge and the diamond.

In Derailment 5 an old turnout was removed from the CWR track, and a new one was installed and surfaced in cold weather. Too much rail may have been added because of contraction after the CWR was cut, resulting in a lowering of the neutral temperature. This was a facing point turnout for trains on a descending grade; therefore longitudinal rail creep, impeded at the turnout, would further decrease the neutral temperature in the approach to the turnout. Several warm days occurred between the time of the track work and the derailment, but no trains operated during those days. The first train over the track during the heat of the day, in a heavy dynamic braking mode, derailed just ahead of the switch of the turnout because of an apparent buckle.

Derailment 12 involved a situation in which relay CWR was installed 2 to 3 years before the derailment, which occurred in July at an ambient temperature of 98°F. During the previous winter numerous field welds had been made, in which rail plugs were added to remove poor and defective sections of rail. The cutting of the rail occurred at low temperatures, and evidently no allowance was made for the rail's contracting. Thus, too much rail was added, lowering the neutral temperature. It was also learned that at the time the CWR was laid to replace the jointed rail, the alinement and surface were irregular. The track was later surfaced and lined. This would have had the effect of adding more rail and would have further reduced the neutral temperature, even if the rail had been laid at the desired temperature for the area.

Several examples are similar to this one, in which CWR was installed at the desired neutral temperature, but with irregular alinement and surface. When the track was later straightened by lining and surfacing, buckling occurred during hot temperatures. Some carriers do not address this problem in their instructions and do not correctly adjust the rail after it has been cut during cold weather.

Five of the derailments took place near the ends of open-deck bridges. As previously discussed, this is a critical location, at which longitudinal rail creep is impeded and a lower neutral temperature can be expected. Whenever this track is disturbed in hot weather, problems should be anticipated, as in Derailments 2, 11, and 13.

In Derailment 2, a slow order was placed at the time of disturbance but was later lifted. Five days later, in extreme heat, the track at the end of a bridge buckled under a train. In Derailment 11, no rail anchors were found on the 670-ft open-deck trestle, and evidence showed up to 2⅝ in. of longitudinal rail movement on the bridge. This expansion across the bridge at a high temperature would have caused longitudinal creep and high compressive forces at the end of the bridge where the track had been disturbed the month before the derailment. The railroad had no specific instructions about anchoring on bridges with CWR over 300 ft. Some allowance has to be made in these cases to account for longitudinal movement. The railroad later applied rail anchors across this trestle. Each structure must be evaluated by bridge specialists to determine the best method of handling rail expansion for that particular structure. In Derailment 13, the track was surfaced at the approach to a 164-ft open-deck bridge with rail anchors just hours before the derailment. No slow order was in place. The bridge in this case impeded rail creep that had caused a lowering of neutral temperature both in front of and under the train that derailed.

Derailment 9 involved laying the outside rail of a curve with CWR and removing jointed rail. The inside rail remained as jointed rail. The rail was laid at cool temperatures in March, and instructions for heating the rail were not followed. Rail anchors were not added to the inner rail, so anchors were not on the same ties as those installed on the newly laid CWR, reducing the rigidity of the track structure. The track buckled under a heavy train at a high temperature in June.

Over the past 15 years instructions for controlling the behavior of CWR have improved from an annual spring letter from a chief engineer stating, "Don't let the track buckle," to 50-page booklets of instructions for almost every type of situation. The question remains whether some railroads are still just beginning to give instructions and training to personnel in controlling the behavior of CWR. At a minimum, every railroad should have clear instructions regarding slow orders, laying and adjusting CWR, staking curves, anchoring rail, cutting and welding CWR, handling rail pull-aparts in winter, and taking care of rail expansion on structures. Furthermore, a training program must be in place to ensure that personnel involved with CWR maintenance understand the application of these instructions.

A track foreman may not understand the physical principles involved or exactly what is meant by desired neutral temperature, but he does understand that if a piece of irregular track is lined and surfaced, there may be too much rail in that track. Also, if a piece of rail is removed, at a minimum, the same amount of rail must be replaced. The rail may not be adjusted to the desired neutral temperature, but conditions will not be worsened.

How is the effectiveness of the various schemes to control CWR summarized? Experience has shown what went wrong and what should have been done to prevent derailments. There has been an improvement and at least a reduction in derailments caused by track buckling.

If the instructions of several different railroads are reviewed collectively, it is found that most of the problems addressed in this paper are covered to some extent by at least one railroad. Each railroad is urged to take the best instructions from the others to cover the whole spectrum of potential situations. After this has been done, the challenge remains to make sure the people doing the work are trained to understand and follow those instructions.

Lateral Track Stability: How Santa Fe Railway Achieves It Today

HERBERT G. WEBB

The Santa Fe Railway has been successful in controlling continuous welded rail thermal stresses that could lead to structural stability failures. The railroad's maintenance engineers take the company rules and guidelines seriously and follow them to the best of their ability. The railroad depends on the first-line supervisors to know the rules, know their territories in relation to possible thermal stresses in the rail, and to ensure that all who work on the welded rail follow the rules when the rail or ballast section is disturbed. Other contributing factors to Santa Fe's success are the adherence to territorial laying temperatures, anchor maintenance, ballast shoulder maintenance, scheduling of maintenance work, ballast compaction, slow order instructions, hot weather patrolling, management allowance of cutting of the rail, and train operation training and handling.

The focus of this paper is the success of the Santa Fe Railway in preventing lateral track stability failures. The author believes the subject to be important and believes that the practices of the Santa Fe might help other railroad maintenance engineers. Only 45 structural kink derailments occurred on the Santa Fe Railway from 1979 through 1988, an average of 4.5 per year. Of these, only 13 occurred on the main lines, and only 22 were on welded rail, an average of 2.2 per year. But even this small number is too many. Derailments are expensive losses for railroads and can have a detrimental effect on their profitability. The average cost per derailment has been $140,000, with one incident of $1.5 million (see Table 1).

Particular attention is paid to the following practices on the Santa Fe in order to achieve lateral track stability on welded rail.

1. Quality maintenance supervision,
2. Territorial target laying temperatures,
3. Anchor maintenance,
4. Ballast shoulder maintenance,
5. Maintenance operations,
6. Ballast compaction,
7. Slow orders,
8. Hot weather patrolling,
9. Cutting and welding, and
10. Train operations.

No one set of rules or guidelines fits the entire railroad system. The climate, track geometry, grade, train operations, and ballast conditions vary greatly from one division to the next and in some territories from one roadmaster to the next.

The Atchison, Topeka and Santa Fe Railway Company, 224 S. Michigan Ave., Chicago, Ill. 60604-2401.

The key personnel in maintaining lateral track stability are the first-line supervisors—the roadmasters or track supervisors. They must see that all rules are followed in their territories. They must ensure that maintenance procedures are followed by the foremen of all rail maintenance operations, from the initial laying of the rail to surfacing, curve relays, ballast maintenance, changing out single defective rails, and the multitude of other maintenance operations of the track section. First-line supervisors must ensure that all who work on the track structure understand what precautions must be taken to protect the delicate status of lateral track stability.

QUALITY MAINTENANCE SUPERVISION

The track structure supervisors—the people responsible for all maintenance performed on the track structure—must be fully knowledgeable of all company rules, instructions, guidelines, and territorial conditions that may affect the lateral stability of the track. They must pass this experience and knowledge to all foremen in the territory to ensure that they understand all precautions to be taken.

The supervisors must understand train operations in relation to slow orders that they or their staff may place in relation to the geometry of the track. It is important to know where there is "tight" rail and where maintenance work was done in cool or cold weather. They must be able to recognize unstable sections of track and must have management's authority and commitment to cut the rail when it is necessary to relieve excess thermal stresses.

TERRITORIAL TARGET LAYING TEMPERATURES

On the Santa Fe Railroad, a chief engineer's standard designates welded rail laying target temperatures for each subdivision in the entire system (Figure 1). These rail laying temperatures are strictly adhered to during rail relay operations. Rail heaters are used to ensure that the rail has been properly expanded or elongated in order to achieve an operational neutral rail temperature for the geographical area. It is very important, and is stressed to the rail laying supervisor, that the rail must be expanded or elongated, not just heated. Records are kept on the rail relay to ensure that the rail was laid at the designated temperature. The rail is spiked and anchored as quickly as possible behind the small, portable heaters used on curve relay gangs and other rail replacement operations.

TABLE 1 DERAILMENT STATISTICS

YEAR	MAINLINE CWR	MAINLINE JT	NON-MAINLINE CWR	NON-MAINLINE JT	TOTAL
1979	0	0	0	3	3
1980	4	0	7	2	13
1981	2	1	0	4	7
1982	0	0	2	1	3
1983	1	0	2	2	5
1984	0	0	0	2	2
1985	0	0	0	4	4
1986	1	0	1	1	3
1987	0	0	0	0	0
1988	1	1	1	2	5
Total 10 Years	9	2	13	21	45

ANCHOR MAINTENANCE

Every other tie on welded rail is box anchored. Anchor squeezing applicators are used to ensure that the anchors are tight against the tie. All ties in turnouts to which the applicators can be physically applied are fully box anchored. Sixty ties in both directions from all track joints in welded rail territory are also fully anchored.

In all major maintenance operations, such as mechanized tie renewal or ballast undercutter cleaner programs, missing or lost anchors are replaced. Anchor squeezers are used on major surfacing projects to ensure that the anchors are tight. Automatic squeeze tamping operations tend to move the crossties tightly against the anchors on one side, thus leaving the other two anchors ineffective. Anchor maintenance is an important part of maintaining lateral track stability.

BALLAST SHOULDER MAINTENANCE

The Santa Fe Railway ballast shoulder standard is somewhat less than that of many major railroads—6 in. on tangent track and 12 in. on the high side of curves of 2 degrees and more (Figure 2).

Determining the amount of necessary ballast shoulder is always a difficult decision for the maintenance engineer. A good solidified ballast section assists in lateral stability, whereas too much ballast prevents good drainage.

MAINTENANCE OPERATIONS

A number of precautions are taken on the Santa Fe Railway to preserve lateral stability during track maintenance operations. Every effort is made to perform maintenance at a temperature at which the lateral track stability will be least disturbed. In many cases this effort is not successful because of the size of the railroad and the economics of gang scheduling.

Maintenance personnel try to recognize areas of tight rail. In some cases local division supervisors will cut the rail and let it run before the programmed maintenance operation. Many times a welder is placed with a tie gang or ballast cleaning operation as an added precaution so that if tight rail is found, it can be cut and welded immediately. A disturbed track slow order is also placed.

Another company rule is never to add rail when cutting in or replacing a rail for any reason. The rail section is not tightened in such operations as lining in curves or making large surfacing raises through short sags in the grade. If these operations must be accomplished, welders are available to cut and weld.

BALLAST COMPACTION

Ballast crib and shoulder compactors are used behind all major surfacing operations. This compaction provides the approximate equivalent of 3 or 4 days of train operation in restoring lateral stability of the track. The slow order can be removed much sooner and in many cases does not need to be placed at all. The compaction replaces a good portion of the lateral track stability that existed before the maintenance operation.

SLOW ORDERS

The following slow orders are placed on all disturbed track when the ambient temperature is 80°F or higher or when the rail temperature is above the adjusted rail laying temperature.

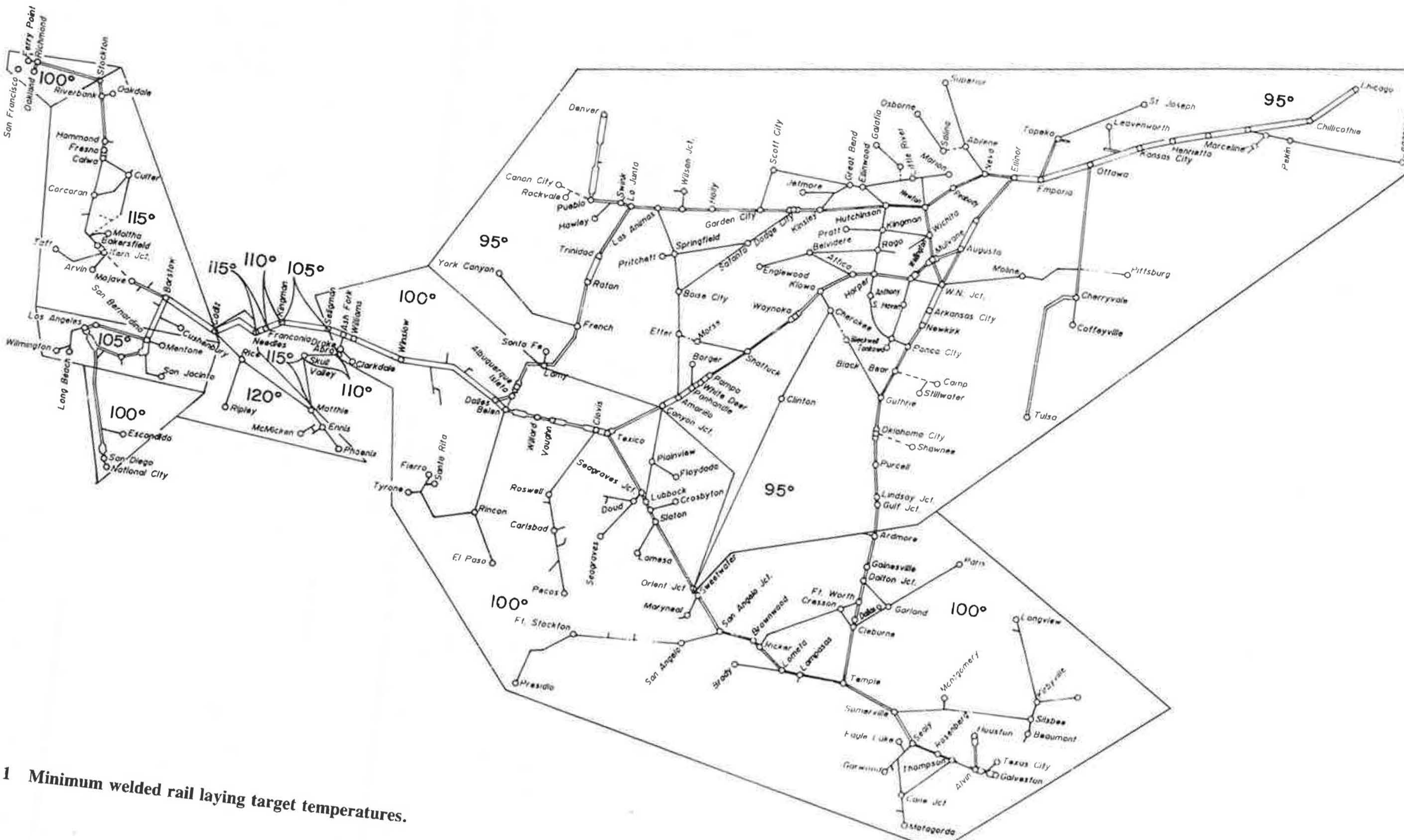
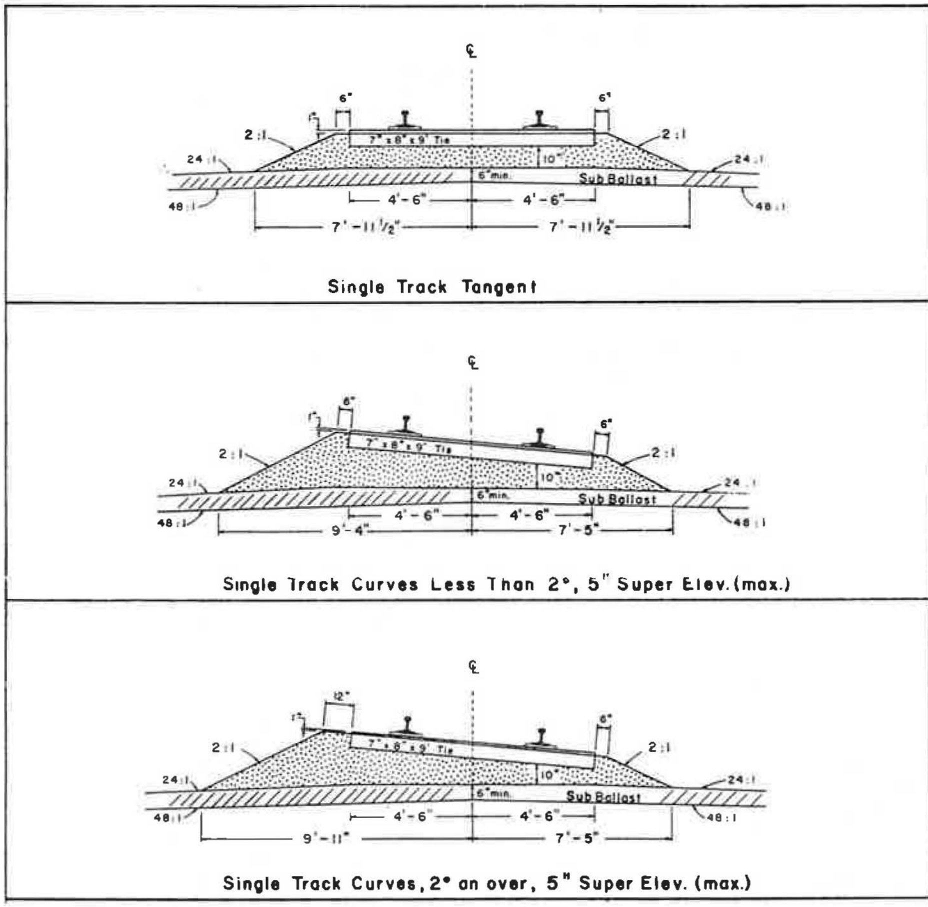


FIGURE 1 Minimum welded rail laying target temperatures.



SINGLE TRACK

	℄ of Track to toe of slope of ballast		Cubic yards per mile
Tangent Track	7'-11 1/2"		3487
	OUTSIDE	INSIDE	
Track Curves Less Than 2°	9'-4"	7'-5"	4321
Track Curves Greater Than Or Equal To 2°	9'-11"	7'-5"	4600

NOTES

1.-Top of ballast shall be finished 1" (inch) below top of tie where applicable.

2-Yardage of ballast shown on this sheet has been computed on the basis of 3250 sawn cross ties, 7" x 8" x 9', per single track mile. An allowance of 15 % for shrinkage is included.

3-For sections on curves the distance from centerline of track to toe of slope of ballast is computed for superelevations of 5" (inches). This distance will vary with each amount of superelevation, and must be computed in each case.

4-Ballast quantities for sections on curves are computed for superelevations of 5" (inches) and shall also be used for all curves with less than 5" (inches) of superelevation.

5-Chief Engineer's authority must be obtained before using a ballast section other than the 10" (inch) section shown.

6.-Refer to C. E. NO 5670, sheet 1 of 2, for cross tie length and assignment.

FIGURE 2 Ballast shoulder standard.

- On main track with 30 million gross tons (MGT) or more, a speed limit of no more than 30 mph should be placed for at least 24 hr.
- On main track with 20 to 30 MGT, a speed limit of no more than 30 mph should be placed for at least 48 hr.
- On main track with 20 MGT or less, a speed limit of no more than 30 MPH should be placed for at least 72 hr.

Disturbed track is the result of any maintenance operation that causes the rail to be cut or any operation that disturbs the ballast section. A roadmaster or designated local maintenance officer has the freedom to place a slow order at any location that does not have the lateral stability necessary for full train operations. If the ballast section is not standard or if the track raised was not fully compacted or stabilized, additional orders might be placed.

When it is necessary to perform maintenance that disturbs the track under an ambient temperature below 80°F or a rail temperature below the adjusted rail laying temperature, the foreman of the gang that completes the work must check the crosslevel and alignment of the disturbed track and place a speed restriction, if necessary, to provide for the safe operation of trains and engines.

Before the release of a slow order, the roadmaster or his designated representative must inspect the track even if the prescribed time period required in the disturbed track order has elapsed.

When a ballast compactor is used in conjunction with a surfacing operation, and inspection by the foreman indicates that standard ballast section, alignment, and surface are proper, it is not necessary to place any of the above speed limits.

It is important to recognize track geometry locations where care must be taken in placing orders. The supervisor must understand train operations and dynamic brake applications to ensure that an order is not placed at a location at which it is impossible for the train engineer to comply without causing large stresses in the track.

HOT WEATHER PATROLLING

The roadmaster, assistant division manager of maintenance, or the assistant superintendent of maintenance decides during certain times of the year to perform hot weather patrolling. The decision depends on their knowledge of the railroad and the climatic conditions of that territorial location. It must be recognized when these prolonged hot periods are a definite danger to the lateral stability of the track. Seven-day patrolling is initiated on the railroad during these conditions, and particular attention is paid to the track in the late afternoon hot periods.

Another hot weather tool used on the Santa Fe Railway is running branch line grain trains and sometimes normal freight trains at night whenever possible. These nighttime operations are mostly on branch lines on which ballast or maintenance problems are known to exist.

CUTTING AND WELDING

One of the typical phrases that can be heard by Santa Fe maintenance supervisors is, "When in doubt, cut, cut, cut." It may sound strange or even funny, but it is important to the lateral stability of welded rail. First-line field supervisors have the freedom and the responsibility to decide where and when to cut welded rail in order to relieve thermal stresses that the supervisor believes pose a threat to lateral track stability. The newly cut joint must be replaced with a field weld as soon as possible. Track joints should not exist in welded rail.

TRAIN OPERATIONS

An important consideration that tends to be forgotten is the effect on lateral stability of train operations. With the advent of dynamic braking, where 4,000- to 8,000-ton trains are being braked at the front end of the train, large lateral forces at rather short concentrated areas are exerted on the track structure. In many cases these forces occur at weak sections of track next to road crossings, turnouts, or other locations that maintenance engineers are trying to protect. The Santa Fe Railway has established an educational program on train handling for train engineers. The program covers such topics as the forces placed on track when dynamic brakes are used. In addition, the assistant division managers of maintenance take the engineer training program. A result of this training has been a better understanding of what the engineer can or cannot do when he approaches a slow order situation in relation to the handling of air or dynamic brakes and the geometry of the track.

The education of all concerned in the proper use of dynamic brakes has been a big help in controlling structural kink derailments and maintenance problems on the Santa Fe Railway.

SUMMARY

In summary, maintenance engineers on the Santa Fe Railway try to follow the established rules for maintaining lateral track stability of welded rail. A great deal of responsibility is given to the first-line supervisors to establish, preserve, and protect lateral track stability of continuous welded rail. The supervisors are given the rules, guidelines, instructions, and tools needed to accomplish this task, but it is still up to them and their foremen to actually perform all maintenance operations within those instructions and guidelines.

Maintenance engineers are rather proud of the railway's record in preventing structural kink derailments over the last few years. However, thermal stresses in welded rail are fickle. It seems that the forces on the trackage keep changing, which makes the job of maintaining the lateral track stability of that trackage an ever-changing one. Better methods, rules, equipment, and procedures must continually be developed to assist the first-line maintenance supervisors in their job of maintaining that lateral stability.

Methods and Procedures for Laying and Maintaining Continuous Welded Rail To Attain Lateral Track Stability

BRUCE G. WILLBRANT

Lateral track stability is attained by proper methods and procedures for laying and maintaining continuous welded rail (CWR). CWR must be anchored at or adjusted for a rail temperature of 95°F or higher either by mechanical heating or by natural temperatures. After CWR has been installed, it should not be raised or disturbed at rail temperatures higher than the anchored or adjusted rail temperature except when necessary precautions are taken. If the track buckles while it is being worked because of expansion due to temperature, it must be cut, adjusted, and properly tamped. When thermite welding is performed, a defective rail is changed, or a plug is installed, precautions must be taken to not add rail to the track. Additional rail creates undesirable compressive forces when increased temperatures cause elongation of the rail. The proper methods and procedures to attain lateral stability have been generated from the use of CWR in track for the past 30 years.

Consolidated Rail Corporation (Conrail) was made up of several Northeast railroads on April 1, 1976. Each railroad had its own policies and procedures for performing track maintenance. Continuous welded rail (CWR) has been in track on the railroad for more than 30 years. An attempt will be made to point out what has been done through the years to arrive at present methods and procedures.

In the early years CWR was not heated, and it was not laid if the temperature was below 40°F. Thus, rail was only laid between April 1 and October 31. It was thought that if the rail was laid at temperatures between 60° and 80°F, no later adjustment to compensate for temperature would have to be made unless the need was clearly demonstrated by some condition in the track. Buffer rails were used at the ends of full-length strings to allow for some contraction and expansion. When the rail was laid at temperatures below 60°F, it was necessary to readjust and install a shorter buffer rail during the first hot weather spell. Likewise if the CWR was laid and anchored at a temperature greater than 80°F, the buffer rail would have to be readjusted during cooler weather. As can be seen in many cases, buffer rails were being changed in the spring and fall because of the inability to lay or maintain rail at the mean temperature.

When buffer rails were not used and the rail temperature was over 80°F, the rail was laid in compression by bumping; when the rail temperature was 60°F and under, the rail was laid in tension by pulling. As can be seen in the initial rail laying procedures, there was very little control over temperature, and if the rail was not laid in the 60° to 80°F range, other precautions had to be taken. As the number of miles of CWR increased, it became very difficult and costly to continue to readjust and change buffer rails. In the mid 1960s it became apparent that for proper installation and maintenance rail had to be mechanically heated to a desired uniform temperature, which was accomplished by introducing heat from one end of each string to the other in the direction of rail laying. The number of inches that the string was to be expanded for the rise in temperature was calculated and the gap was set for the expansion and closed. This also allowed both rails to be anchored at the same temperature, which is a significant factor in preventing buckled track.

A pull-apart caused by a drop in temperature was considered more tolerable than buckled track caused by a rise in temperature. A train has a much better chance of traveling over a pull-apart than buckled track without derailing, so it was determined that when the CWR was heated, it was to be anchored at 85°F. Also, signal systems give protection when pull-aparts occur that interrupt the track circuit. In the late 1960s CWR strings were field-welded together, which, because of the elimination of bolted joints, reduced the potential for pull-aparts.

In the early years CWR was not disturbed for maintenance work in the months of July and August, but as the amount of CWR in track increased, maintenance became necessary regardless of the temperature. In the mid-1960s, when the air temperature exceeded 80°F, the track was worked during the early morning hours and protected by a temporary 30-mph slow order until the rail cooled in the evening. When it became necessary to work CWR in warm weather months, proper precautions had to be taken, such as the following:

- A full ballast section had to be maintained at all times.
- When ties were installed, the ballast removed from the tie ends had to be kept to a minimum and backfilled every night.
- During tie installation the track raise was kept to a minimum; both rails were raised simultaneously and a crosslevel was maintained at all times.
- Anchors removed for tie installation were reinstalled immediately after spiking.
- When the track was raised for surfacing, the raise was kept to a minimum, both rails were raised simultaneously, and a crosslevel was maintained at all times.
- When track was tied and not tamped, the first train was restricted to 30 mph, and the track was inspected by the track supervisor before the slow order was lifted.

Consolidated Rail Corporation, Room 1634, Six Penn Center Plaza, Philadelphia, Pa. 19103.

- All cribs were filled completely the day the track was tamped, and final dress was completed as soon as possible.
- The final ballast section required all cribs to be full to the top of the tie and at least 12 in. beyond the end of tie before sloping off to the subgrade.

Experience had shown that a track that is shy of ballast in the cribs or that was raised excessively had a definite tendency to kick out or buckle. Also, rail in embankment cuts retains more heat than rail on fills where the air is free to circulate. All such locations of restricted air circulation were observed closely during periods of high temperature. Many of the these procedures are still followed today.

Through the years of working CWR in warm weather, maintenance engineers have encountered problems with lateral stability at the ends of restrained areas such as road crossing, bridges, station platforms, turnouts, and the like. In the mid-1970s it was decided to heat and anchor the rail to 95°F. Pull-aparts had been successfully eliminated by field welding, but the railroad was still experiencing some trouble with buckled track.

Experience has also shown that if a track buckles, it must be cut and readjusted because it is likely to buckle again if it is not adjusted properly. Work crews tend to realign the buckled portion without cutting the rail, a procedure that is not tolerated. Readjusting the rail by cutting out the buckled portion has reduced the potential for buckling.

In the early days of Conrail, CWR use was restricted on curves of more than 6 degrees. This instruction was later changed to allow the use of CWR on all curves, but they were monitored for any indications of movement up and out of the plates. If tipping occurred, the rail expansion was adjusted and base clips were installed to prevent overturning. During the last few years, due to types of traffic and tonnage, problems have been experienced with CWR overturning on curves. Elastic-type fasteners on lines with severe curvature, grades, and tonnage are now being installed.

Track patrols are conducted 7 days a week when the air temperature is above 90°F or below 20°F. The patrols are operated at the discretion of the division engineer. Instructions for laying and maintaining CWR are uniform over the system because there is no significant temperature variation across the railroad to warrant different instructions for each specific area.

There have been problems with self-jacking lining tampers when track is surfaced on curves in cold weather. There is a tendency to line curves to the inside because it is the path of least resistance. This happens without the operator's knowledge unless reference points are established and monitored to ensure that the curve is not being lined to the inside. Production gangs are normally shut down from late October or early November until early April, which helps alleviate some of the problem of curves being lined to the inside. If curves must be worked in cold weather, reference points must be established to ensure that the curve is not lined exclusively to the inside.

Methods and procedures have been adopted during the past 30 years to reduce the potential for buckled track during work on CWR under various temperature ranges. The procedures have helped reduce the amount of buckling but incidents still occur where there are curves, grades, and helper units. These incidents are caused by improper train handling and not the track. Precautions are taken by placing slow orders on newly worked CWR track so that the braking of the train will not affect the unstable track.

Incorporated in training schools and seminars for supervisors is a review of the methods and procedures for maintenance on CWR. Because of problems with buckled track through the years, supervisors are made aware of the consequences of failure to follow procedures. Each time a derailment occurs because of buckled track, maintenance methods and procedures are evaluated to see if they are adequate for present-day operation.

Lateral track stability is attained by following proper methods and procedures for laying and maintaining CWR. Conrail's present methods and procedures are outlined in this paper.

Lateral track stability starts with the proper rail laying procedures. CWR must be anchored at or adjusted for a rail temperature of 95°F or higher. When the rail temperature is lower than 95°F, a heating device is used for expanding the CWR to make the proper adjustment. When CWR has been anchored at a temperature below 95°F and not adjusted for temperature during the rail laying operation, it should be adjusted as soon as weather conditions have brought the rail to a temperature of 95°F or higher. The anchored rail temperature and length of adjustment must be recorded and retained for future reference when the involved stretch of rail is worked.

ADJUSTMENT BY MECHANICAL HEATING

Rail may be expanded after it has been laid in the tie plates and before or after spiking, but it must be expanded before it is anchored. CWR should be heated so that the expansion is introduced from one end of each string to the other in the direction of rail laying. The number of inches by which each CWR string should be expanded during the rail laying operation may be determined by calculation or from an existing table (see Figure 1). A gap equal to the amount of expansion needed for each string of CWR should be provided between the end of that string and the end of the next adjacent string. A minimum of 10 ties should be box anchored on the near end of the adjacent string to hold the string in place and to avoid closing the expansion gap in the reverse direction, which would improperly adjust the string being heated. Heating should start at the beginning of the first CWR string and be applied steadily until the required expansion has been obtained at the end of the string. Uniformity of expansion is to be controlled by marking each quarter of the string and introducing expansion as follows:

- Quarter point: one-fourth of total required expansion,
- Half-point: half of total required expansion, and
- Three-fourths point: three-fourths of total required expansion.

Quarter points should be marked on the rail and the tie plate to ensure that the amount of expansion is accurately determined. The tie plate used as a reference point must be one that is spiked, so that it will not move as rail expands. If

MW 155 R3 11-81
Printed in U.S.A.

CONSOLIDATED RAIL CORPORATION
CONTINUOUS WELDED RAIL
RECORD OF RAIL LAYING

Region ______________________ Division ______________________ Line ______________________

Date ______________________

Table for Adjustment of CWR For Temperature Change

Measured CWR Temperature	Length of CWR in Feet					
	950′—1049′	1050′—1149′	1150′—1249′	1250′—1349′	1350′—1449′	1450′—1550′
111—120	+2	+2	+2	+2	+2	+2
101—110	+1	+1	+1	+1	+1	+1
90—100	0	0	0	0	0	0
80—89	1	1	1	1	1	1
70—79	2	2	2	2	2	2
60—69	2	3	3	3	3	4
50—59	3	3	4	4	4	5
40—49	4	4	5	5	5	6
30—39	5	5	6	6	7	7
20—29	5	6	7	7	8	8
10—19	6	7	7	8	9	9
0—9	7	8	8	9	10	11
-10—-1	8	9	9	10	11	12
-20—-11	9	9	10	11	12	13

NOTE: All adjustment figures are in inches.
In the event temperature or lengths do not fall within table coverage, Local Supervisor will compute adjustment in compliance with Paragraph 119.5, MW-4, dated March 1, 1977.

Rail Record

String No.	Track No.	E or N W or S	Rail	M.P. From	M.P. To	Rail Temp.	Adjustment From Table	Quarter Point Adj. 1st	Quarter Point Adj. 2nd	Quarter Point Adj. 3rd
1										
2										
3										
4										
5										
6										
7										
8										
9										
10										
11										
12										

Signed ______________________

Title ______________________

FIGURE 1 Conrail record of rail laying for CWR.

the first half of the heated CWR string does not have the required expansion at each quarter point, the heater will back over the heated portion without applying heat and then reheat the rail until the necessary expansion is obtained. As heating progresses, a minimum of 1 anchor per 39 ft of rail should be applied on the side of the tie that will prevent the rail from losing expansion. At the end of the completely expanded string, a minimum of 10 ties should be box anchored immediately after the gap is closed to hold the expansion. The entire CWR is to be anchored as described or per standards before trains are permitted to operate over it at timetable speeds. CWR is to be anchored in both directions by box anchoring as follows.

Every Tie (Full Boxing)

In the following areas, every tie is box anchored:

- Curves of 3 degrees and more;
- At each bolted end of a CWR string for 200 ft, except where CWR strings are butt-welded together in the field, in which case every other tie is box anchored;
- Adjacent to each side of track crossings for 200 ft;
- Adjacent to each side of open floor bridges for 200 ft;
- Adjacent to each side of public and private road crossings for 200 ft;
- Through turnouts laid with CWR to the extent practicable, and for 200 ft adjacent to switch ties and each end of turnouts through which CWR extends; and
- Through CWR strings less than 400 ft long.

Every Other Tie

In the following areas, every other tie is box anchored:

- Through the remainder of each CWR string where full boxing is not specified above; and
- Across open floor decks on timber and steel structures where blocking has been placed between bridge ties and the deck is properly fastened with hook bolts.

ADJUSTMENT BY NATURAL TEMPERATURE

When it is necessary to adjust CWR already in track, the required increase or decrease may be found by taking the difference between the desired and the recorded temperature of each string of CWR and calculating the amount of adjustment or by using an existing table. All rail anchors must be removed from strings of CWR requiring adjustment to permit the desired expansion or contraction. Tie plates should be tapped with a hammer or mechanical device to free the rail. All rail anchors must be reapplied immediately after the desired change in rail length has been obtained. Where numerous strings need adjustment, it is desirable to make adjustments for three or four strings at a time, if possible. For this purpose, a rail cut should be made near the center of the adjusted area. When adjoining CWR strings are connected directly by a bolted rail joint, the adjustment for either compression or tension should be made by cutting out the drilled end of each CWR and field welding in a rail of required length. Where CWR strings are field butt-welded together, the adjustment may be made by cutting and butt welding in a piece of rail.

REPLACEMENT OF DEFECTIVE RAIL OR WELD

In order to avoid addition of rail when thermite welding is performed, a defective rail is changed, or a plug is installed, the following procedure must be used if the rail temperature is less than 95°F. During thermite welding, the required gap must always be obtained by cropping the ends of the rail and the gap maintained by using a rail stretcher, if necessary. When a defective rail is changed or a plug is installed, the length of rail to be replaced must be measured before removal and the piece to be installed cut to the same length. The gap that remains after installing the piece of rail must be closed by heating.

PROCEDURES FOR MAINTAINING AND WORKING CWR TRACK

After CWR has been laid and adjusted, proper maintenance procedures must be followed to ensure lateral stability of the track. The track should not be raised or otherwise disturbed at rail temperatures higher than its installation or adjusted rail temperature except when the necessary precautions are taken. The following work should not be performed unless measures are taken to protect the track.

- Out-of-face track raising,
- Heavy tie renewals (with or without raising),
- Extensive lining or disturbing of the ballast section, and
- Smoothing or lining where more than five consecutive ties are loosened from their tie beds or where more than five consecutive or intermittent ties are loosened from their tie beds in any 39-ft length of track.

Rail Temperature Equal to or Below Installation or Adjusted Temperature

The following requirements apply to maintenance performed on track whose rail temperature is no higher than the installation temperature or the latest adjusted rail temperature:

- When CWR track is raised, the height of the raise should be kept to the minimum necessary to obtain a good surface but should not exceed 1½ in. If a higher raise is needed to meet a required profile, additional raises should be made with enough elapsed time between raises for the track to become sufficiently settled by the passage of trains to ensure stability at timetable speed. If the track is undercut, the above will not apply if the rail is cut and adjusted.
- Both rails should be raised simultaneously in CWR track, and a crosslevel should be maintained at all times. Raising without immediately and fully tamping all ties should be avoided.

- When ties are renewed, no more than three consecutive ties or eight ties per 39-ft section of rail should be renewed in any one pass. If more ties need to be renewed, additional passes should be made.
- Before track is returned to normal service, all ties installed should be rail spiked and tamped; rail anchors should be reapplied and standard ballast section restored. A standard ballast section for CWR should be all cribs full to the top of the tie, 12 in. of ballast straight out from the end of the tie, and a 2:1 slope to the sub-ballast line.
- The temperature at which the rail is worked should be recorded, but should not be considered as the adjusted temperature.
- An appropriate slow order, not to exceed 30 mph, should be placed on all track worked that day. The slow order should remain in effect for 24 hr and until 50,000 gross tons of traffic has passed over the work area. The division engineer should determine through the dispatcher when the minimum tonnage has run over the work area and make arrangements for inspection of the track and a possible increase in speed. If an inspection of the work area reveals no exceptions, the speed of the track should be upgraded to timetable speed.

Rail Temperature Higher Than Installation or Adjusted Temperature

If the measured rail temperature is higher than the installation or the latest adjusted rail temperature, the following procedures apply to the adjustment of CWR before or during maintenance operations:

- The ends of CWR strings out of the tie plates should be disconnected or cut and lined to clear adjoining rail ends.
- All anchors should be removed from the area to be adjusted.
- After the track has been raised, tamped, and lined, rail closures should be made and the CWR adjusted as needed.
- All rail anchors should be reapplied to prescribed standards before the track is returned to normal service.
- A standard ballast section should be restored before the track is returned to normal service.
- In the event work is performed through only part of a CWR string, the entire string should be freed, and the unworked portion of the string should be loosened in its tie plates by operating a heavy self-propelled unit of maintenance-of-way equipment over the unworked portion or tapping the tie plates with a hammer before closure and anchoring.
- The rail temperature of each CWR string that is adjusted should be measured and recorded.
- If the rail is adjusted before or during the maintenance operation, as outlined above, the track may be placed in service with an appropriate slow order not to exceed 30 mph on all track worked that day. The slow order should remain in effect for 24 hr and until 50,000 gross tons of traffic has passed over the work area. The division engineer should determine through the dispatcher when the minimum tonnage has been run over the work area and then make arrangements for inspection of the track and possible increase in speed. If an inspection of the work area reveals no exceptions, the speed of the track should be upgraded to timetable speed.
- If the rail is not adjusted before or during the maintenance operation, a 10 mph slow order should be placed on the work area when the track is returned to service, and the track should be inspected after the first train.
- The 10-mph slow order should remain in effect for 24 hr and until 50,000 gross tons of traffic has passed over the work area. Provided no exceptions are taken after inspection, the order should be upgraded to 30 mph, which should remain in effect another 48 hr and until another 50,000 gross tons of traffic has passed over the work area. The division engineer should determine through the dispatcher when the minimum tonnage has been run over the work area and should then make arrangements for inspection of the track and a possible increase in speed. If inspection reveals no exceptions, the work area should be upgraded to timetable speed.
- When the latest adjusted temperature is unknown and the existing rail temperature is 80°F or above, the instructions for performing work when the rail temperature is higher than the installation or adjusted temperature should apply.

Slow Orders for Track Stabilized by Dynamic Track Stabilizer

Instructions for newly surfaced CWR track that has been stabilized by a dynamic track stabilizer immediately after the surfacing operation are as follows:

- The track should be inspected by the gang supervisor before being returned to service.
- A slow order of 10 mph should be placed on the work area for the first train, or at least 5,000 gross tons of traffic.
- After reinspection of the track, a slow order of 30 mph should be placed on the work area for at least one train, or at least 5,000 gross tons of traffic.
- After another reinspection, a slow order of 50 mph should be placed on the work area for at least one train, or at least 5,000 gross tons of traffic.
- The track should be returned to service at timetable speed after a third reinspection.

Maintenance of Buckled Track

If the track buckles while it is being worked because of expansion caused by temperature, it must be cut, adjusted, and properly tamped using the following procedures:

- Both rails should be cut with a torch at the location of maximum displacement after the track has been lined sufficiently to ensure that all pressure has been removed and to prevent the track from reacting rapidly when it is cut. If the displaced area is near a joint, the joint bars should be removed.
- The cut or uncoupled rails should be aligned, allowing the ends to bypass.
- In order to ensure that the expansion is made uniformly throughout the rail being adjusted, the rails should be marked at 330, 660, and 990 ft from the location where the rail ends are bypassed.

- All anchors should be removed for ¼ mi (1,320 ft) from each side of the location at which the rails have been bypassed in order to properly adjust the rail.
- If the rail temperature is over 95°F, the rail adjustment can be completed. The expansion should be uniformly distributed throughout the 1,320 ft of rail. The distribution can be determined by noting the amount of rail movement at the previously marked locations at 330, 660, and 990 ft from the bypassed ends. Particular attention should be paid to ensure that the rail does not bind on tie plates, spikes, or other obstructions. The tie plates should be tapped as necessary to obtain free rail movement.
- After proper expansion has been attained throughout the 1,320-ft rail, the anchors should be reapplied. The application of anchors should start at the point 1,320 ft from the location where the rails are bypassed and work toward that area. Each point marked on the rail should be checked to ensure that the expansion is being made uniformly throughout the rail. All anchors should be reapplied properly and installed tightly against the ties.
- If the rail temperature is under 95°F, the rail should be heated to obtain the proper adjustment. The procedures to be followed are the same as those outlined for adjusting the rail when the temperature is over 95°F. The rail should be heated from the point 1,320 ft from where the rails are bypassed, and the anchors should be reapplied to hold the expansion as the heater moves toward the rail bypass point. Care should be exercised to ensure that the rail is heated to a minimum of 95°F before the anchors are reapplied.

If the rail temperature is less than 95°F and it is not possible to adjust it immediately to that temperature by heating, the following procedures should be followed:

- The rail should be cut or the splice bars removed at the location of maximum displacement after the track has been lined as necessary to ensure that all pressure has been removed.
- Track and bypass rail ends should be aligned.
- All rail anchors should be removed for 1,320 ft and the expansion should be adjusted, making certain that the rail does not bind on tie plates, spikes, and the like.
- After rail expansion has been adjusted evenly throughout the 1,320 ft, the anchors should be reapplied, making sure that they are all tight against the ties.
- The track should be lined back to proper locations and additional cuts made on the rail as necessary.
- The area adjusted should be protected by a maximum 10-mph slow order until the rail expansion is adjusted to 95°F with or without heating.

A new heat record will be prepared with the new adjusted temperature. It should also be noted on the record that the adjustment was made by use of a heater or by natural temperature change and also that all anchors were removed in order to make the adjustments.

SUMMARY

Conrail's practices and procedures for laying and maintaining CWR have achieved the desired results, but have created problems such as loss of production by gangs working CWR during times of high rail temperature and delay of train traffic while tonnage and time requirements have been satisfied. Despite these problems, Conrail has been successful in eliminating buckled track incidents by following the methods and procedures outlined in this paper.

Effectiveness of Southern Pacific Lines in Controlling the Behavior of Continuous Welded Rail Track

DAVID T. WICKERSHAM

The recent efforts of Southern Pacific Lines in controlling the behavior of continuous welded rail are presented. The three principal causes of track buckling are presented, and standards, instructions, rules, and procedures in effect on Southern Pacific Lines that are used to prevent track buckling are discussed in detail. Southern Pacific Lines' training programs for maintenance-of-way employees and locomotive engineers in preventing track buckling are also presented. Also included in the paper are the chief engineer's instructions on track maintenance to protect against lateral track movement, track buckling, and pull-aparts and Operating Rule 465, Train Handling over Disturbed Track.

The number of track buckling incidents on the Southern Pacific Lines has decreased steadily over the past 6 years even though the total miles of continuous welded rail (CWR) are increasing, as are the average tons per train and the average speed per train. The knowledge learned from experience, from other railroads, and from track research in the United States has resulted in this successful performance.

Southern Pacific recognized early the advantages of CWR and since 1956 has replaced jointed rail with new and cascaded second-hand rail in the highly diversified main lines and other tracks. The lines traverse granite mountains; hot, arid deserts; and humid, marshy swamps. The geometry of the track includes 15-degree curves and 3.5 percent grades. Tracks run through areas where record snowfalls occur and areas that receive the greatest and least amounts of rainfall. Ambient temperatures produce rail temperatures on the Southern Pacific Lines from a high of 156°F to a low of −50°F. CWR is used in all these areas.

A great deal has been learned over the years about how CWR must be laid and maintained. The instructions, procedures, and track standards of Southern Pacific Lines have changed as it has been learned how to better maintain lateral track stability. These instructions are detailed in the chief engineer's instructions for the maintenance-of-way and structures.

Track buckling is the formation of lateral misalignments caused by any one or a combination of the following:

- High compressive forces caused by thermal loads and low neutral temperatures,
- Weakened track conditions due to low track resistance or alignment deviations, and
- Vehicle loads.

Southern Pacific Transportation Co., 400 E. Toole St., Tucson, Ariz. 85701-1899.

The manner in which each of these three causes of track buckling has been addressed is the largest factor that has helped avoid track buckling on the Southern Pacific Lines. These factors are detailed in the chief engineer's instructions, specifically Section 2.8, Track Maintenance Procedures Required for Protection Against Lateral Movement of Track, Track Buckling and Pull Aparts, and Section 2.9, Maintenance of Continuous Welded Rail. Copies of the chief engineer's instructions are available from the author on request.

HIGH COMPRESSIVE FORCES

High compressive forces caused by thermal loads and low neutral temperatures must be prevented by laying CWR at or above the required neutral rail temperature, maintaining that minimum neutral rail temperature, and maintaining track to the required common standards.

Instructions to maintenance employees include a zone map of the Southern Pacific system that specifies the minimum neutral rail temperatures allowed for each geographic area of the system (Figure 1). Neutral temperature is defined as the rail temperature at which the net longitudinal force due to thermal stress is zero and the rail is under neither tension nor compression. Minimum neutral rail temperature is defined as the lowest rail temperature to which CWR is installed and maintained. The minimum neutral rail temperature on the Southern Pacific Lines varies between 90° and 120°F. This temperature was originally based on the following equation:

$$\mathrm{ART} = \frac{(2.1TH + TL)}{3} \tag{1}$$

where

ART = adjusted rail temperature,
TH = highest rail temperature expected locally, and
TL = lowest rail temperature expected locally. (In areas subject to extreme low temperatures, the average low temperature is used instead of the lowest temperature.)

However, during 1988, in certain areas of the system, the minimum neutral temperature was adjusted upward to provide additional protection against track buckling.

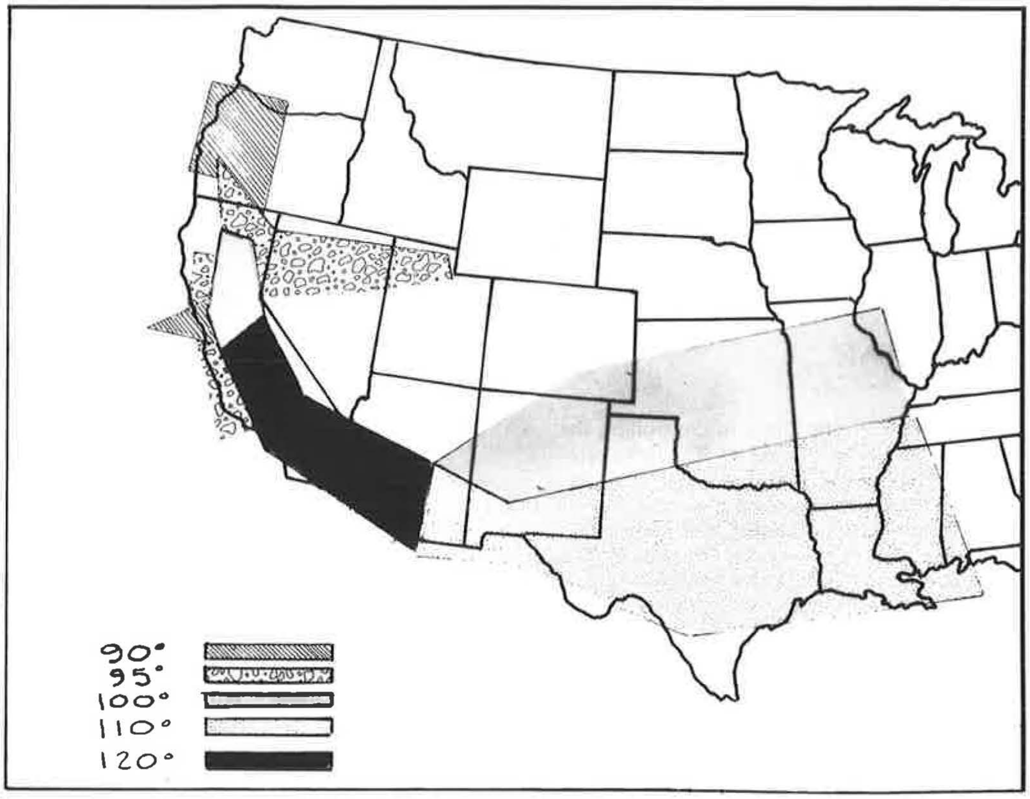


FIGURE 1 Southern Pacific Lines zone map of minimum neutral rail temperatures.

It has been learned from track buckling research and experience that track buckling can occur when rail temperature increases 60° or more above the adjusted rail temperature. In no instance on the Southern Pacific system does the highest rail temperature found in the zone exceed by more than 40°F the required minimum rail neutral temperature identified on the zone map. Instructions require that if the rail temperature difference between laying and current reading exceeds 40°F, the rail must be destressed.

Instructions to maintenance employees include procedures for the proper laying of CWR. Important guidelines in the instructions for controlling proper rail laying temperature include

- Laying rail at or above the minimum neutral rail temperature for the zone, which varies by zone between 90° and 120°F.
- Using a rail heater when laying more than 1,400 ft and the temperature is less than the minimum neutral rail temperature.
- Using a rail heater immediately before anchoring and spiking and in conjunction with a rail vibrator.

The instructions also include procedures for the proper maintenance of CWR. Important guidelines in the instructions for maintaining proper rail temperature include the following:

- When defective rails are replaced, the amount of rail should not be increased.
- Roadmasters are requried to keep a record of all locations that require cutting in a piece of rail, making a weld, or repairing a cold weather pull-apart.
- Identification is to be placed on the web of the rail by the welder that indicates the initials of the welder, the date the weld was made, the actual rail temperature, and the adjusted rail temperature.

A detailed procedure for destressing CWR also is included in the instructions.

Included in the chief engineer's instructions are common standard diagrams used by the track foreman. The important standards pertaining to track buckling prevention include the following:

- Ballast shoulder should be a minimum of 6 in. (Twelve in. is recommended on the high side of curves, areas of poor subgrade conditions, and areas in which track buckling has occurred.)
- The rail anchor pattern should consist of four anchors boxed on every other tie.

District engineers have the authority to add anchors where required, including the approaches to road crossings, bridge approaches, track crossings, and turnouts. In 1981 the rail anchor pattern was changed from four anchors boxed on every third tie to four anchors on every other tie. It was planned to change to the new standard during rail relays, but the change was begun with the tie renewal programs. Approximately 80 percent of the Southern Pacific Lines' welded rail is now anchored to the new standard.

DISTURBED TRACK

Weakened track conditions caused by low track resistance or alignment deviations must be controlled by maintaining sufficient lateral track resistance and maintaining track alignment to close tolerances. Lateral track resistance due to thermal loads becomes a factor when CWR is in compression. Instructions of the Southern Pacific Lines contain guidelines for protection of production and maintenance work on days when temperatures are above 90°F. These guidelines require graduated slow orders to be placed for a minimum of 72 hr on track where various types of maintenance work have been performed for a specified time period. For example, if a production tie gang replaces defective ties out-of-face in track territory with freight speeds of 65 mph (FRA Class 5), the first 24 hr requires a 20-mph slow order, the second 24 hr requires a 40-mph slow order, and the third 24 hr requires a 60-mph slow order.

It has been learned through track buckling research and experience that track resistance decreases as a result of disturbance of the ballast section by work such as track surfacing. It is only through train tonnage, the use of track compactors, or both that track will recover to its maximum lateral resistance. The placement of a slow order on continuously welded track that is in compression after ballast has been disturbed allows for the safe passage of trains while lateral resistance is being restored. Track work that disturbs the ballast section is performed, when practical, on CWR at or below the temperature at which the rail was laid or adjusted. This means that track work that disturbs the ballast is kept to a minimum during the months when track buckling is likely to occur (May through August). However, it is not possible to schedule major tie replacement work around this period. Therefore, steps are taken to destress track that is in compression within these limits, and a track compactor is sometimes used to increase lateral track resistance.

VEHICLE LOADS

Vehicle loads must be controlled by the locomotive engineer's use of good train handling techniques. It has been learned through track research and experiences that track buckling has been induced in areas of disturbed track by forces generated by train handling. Instructions to locomotive engineers require use of good train handling techniques in areas of disturbed track and are detailed in Operating Rule 465, as follows:

> OPERATING RULE 465
> Train Handling Over Disturbed Track
> When a train order is received containing the following wording, "BETWEEN (Milepost) AND (Milepost) BE GOVERNED BY RULE 465", engineer must handle the train so that track and structures within specified limits are subjected to a minimum of train handling generated forces.
>
> Adverse forces are imparted to track and structures as a result of excessive speed, harsh slack adjustments, moderate to high draft or buff forces and/or heavy train braking.
>
> These forces are substantially reduced when the engineer controls speed, allows power to drift, makes no slack adjustments and uses no automatic brake while train is passing through the restriction.
>
> As near as practical the engineer will use train handling techniques that reduce adverse forces by making power and brake adjustments prior to or following the restriction and by carefully controlling speed, use of automatic brakes and slack adjustments while train and engines are passing over the restriction.

Instructions to maintenance-of-way employees require issuance of a train order to comply with Operating Rule 465 to cover unstable track segments where buff and draft forces in train handling could induce track buckling. It is issued in addition to slow orders required by other instructions.

TRAINING

The main factor in the reduction of track buckling derailments on the Southern Pacific Lines has been ensuring that the rules and instructions for the prevention of track buckling are followed by employees performing the work. Track employees are trained by division managers, who use videotapes and other educational tools to discuss the reasons why track buckling occurs. Classes for key employees involved in prevention of track buckling are held by district engineers in early spring and early fall. These key employees include roadmasters, track inspectors, track foremen, bridge foremen, and welders. Work procedures to be followed during the summer months are discussed in detail at the spring meeting. Important topics covered include

- Inspection frequency during hot weather (minimum of three times per week in Class 4 and 5 track with 20 million gross tons or more annually; daily during peak temperatures);
- Warning signs to which inspectors should be alert, such as an unusual "wavy" appearance in tangent track, shifting of rail in plates or plate movements on ties, rail lifting in plates, and so forth.
- Required procedures to be followed when track work is done under temperatures of over 90°F;
- Proper application and use of Operating Rule 465; and
- When and how to destress CWR.

During the fall meeting, work procedures to be followed during the winter months are discussed in detail. Important topics covered include

- Proper procedure for repairing defective and broken rails and rail pull-aparts,
- Record-keeping requirements for repairing defective and broken rails and rail pull-aparts,
- Occurrence of track shifting caused by track surfacing work and its effect on lowering neutral temperature, and
- Record-keeping requirements for curves that have shifted.

It is stressed that all work must be done the right way the first time. However, it is recognized that conditions develop that require the addition of rail to the track to repair a pull-apart. These conditions could result from the unavailability of manpower when the pull-apart occurred, broken hydraulic rail expanders, or limited on-track time. Foremen are instructed that when this occurs, the amount of rail added as a result of the pull-apart must be recorded and reported to their supervisors. The next available work period must be utilized to make the proper repair.

To assist district engineers in training their employees, with the permission of the CSX Transportation Company, videotapes *Prevention of Track Buckling* and *Track Maintenance Procedures for Destressing Continuous Welded Rail* are provided.

Locomotive engineers are trained by the road foreman of engines in the proper procedure for handling trains over disturbed track in compliance with Operating Rule 465. A videotape, *Operating Rule 465, Train Handling Over Disturbed Track*, is shown to every locomotive engineer each spring. Discussed in the tape are components of the track structure, the causes of high lateral train-generated forces, and train handling techniques that a locomotive engineer can use on various kinds of terrain to reduce the amount of lateral force placed on the track structure.

CONCLUSION

Track buckling is caused by high compressive forces caused by thermal loads and low neutral temperatures, weakened track conditions due to low track resistance or alignment deviations, and vehicle loads. Because of the inability of anyone to efficiently determine the stresses in CWR accurately, it is believed that the instructions in the form of guidelines for employees to use when performing track work on CWR, the biannual training programs, and the desire to motivate employees to do their work the right way the first time are the best weapons in helping employees prevent track buckling derailments. The best rules, practices, and instructions will never ensure that track buckling will not occur, but, as demonstrated in the successful reduction of buckling derailments, the Southern Pacific Lines and other carriers are winning in the solution of this problem.

Maintenance Procedures for Lateral Track Stability

P. R. OGDEN

The maintenance-of-way procedures and training programs of the Norfolk Southern Corporation for maintaining track stability and preventing buckled track when working with welded rail are described in this paper. The possibility of track buckling is a constant threat. Track alignment problems can be reduced substantially if maintenance personnel stay alert and follow established instructions for maintenance-of-way activities, such as rail laying, tie renewal, surfacing, and smoothing operations. Two steps were taken to improve the effectiveness of prevention of track buckling on the Norfolk Southern rail system. First, instructions for track maintenance activities relative to stability were consolidated into one maintenance-of-way procedure, Standard Procedure 390, Maintaining Track Stability. Second, training programs were established for all first-line supervisors and track foremen to improve their knowledge of why track buckles and how to prevent it.

Lateral stability of continuous welded rail (CWR) concerns everyone involved in track maintenance. It is a timely subject, especially when seasonal change causes the average air temperature to rise each day, and a corresponding rise in rail temperature occurs.

It has been stated and written that the two most outstanding advancements in track maintenance in the last 50 years are the mechanization of maintenance-of-way work and the development of CWR. On the Norfolk Southern rail system, both play a significant role in efforts to control cost and stay competitive in today's transportation market. However, to maximize all the advantages of CWR, track buckling must be avoided. To achieve stability, close attention must be paid to a number of details, which is the topic of this paper.

Two factors that have helped improve the lateral track stability of Norfolk Southern rail stand out. First was the establishment of a written procedure for maintaining track stability with checks and balances to ensure that it is understood and followed—MW&S Standard Procedure 390, Maintaining Track Stability. The second factor is an ongoing program to train field personnel to better understand the problems and solutions associated with buckled track.

BACKGROUND

The use of CWR has been a big part of the maintenance program at Norfolk Southern for a number of years. The first welded rail was laid on Norfolk Southern in 1958. The current 5-year plan is to lay about 600 mi of CWR each year. The Norfolk Southern rail system has over 25,500 track mi, of which 14,910 mi is welded—13,072 mi on the main line and 1,838 mi in yards and sidings.

As the mileage of CWR in track rose in the late 1960s and early 1970s, sun kinks, buckled track, and derailments caused by buckled track occurred.

Track buckles for many reasons. To help understand these reasons more clearly, research has been conducted in the last two decades by the railroad industry, FRA, and others. Greater knowledge on working with welded rail has been gained over the past 15 to 20 years from research and from cooperation among railroads. However, there is no substitute for the experience gained from working through one's own problems and finding one's own solutions.

In the early 1970s, one of the experiences was a derailment that involved a passenger train. A small maintenance gang had spotted ties on a welded rail at a 4-degree curve. It was a hot spring day, shortly after noon. The gang stopped work to allow the train to pass. Contrary to existing instructions concerning tie replacement in warm or hot weather, a slow order was not placed. Consequently, the track where the new untamped ties were located moved under the train, which was running at timetable speed, resulting in a derailment. Several problems relative to the existing instructions and the first-line supervisor's compliance with and understanding of those instructions were discovered in the postderailment investigation.

IMPROVED MAINTENANCE PROCEDURES

To correct these problems and improve overall performance, it was decided that two things had to be done. First, a set of instructions and standards had to be written for working with welded rail that would be clear, concise, and easily understood by all maintenance-of-way employees, including the track foreman. All new and existing instructions related to track stability were consolidated into Standard Procedure 390. The purpose of the procedure is to establish a uniform system for prevention of buckled track. Second, training programs were established to help employees better understand the characteristics of CWR and the caution that must be taken when laying and working with welded rail.

The instructions and the training programs have contributed more than anything else to the prevention of track buckling on the Norfolk Southern railroad.

Standard Procedure 390

The subjects covered in Standard Procedure 390 are as follows:

Norfolk Southern Corporation, 99 Spring Street, S.W., Atlanta, Ga. 30303.

- Track stability factors,
- Track conditions,
- Track inspection,
- Crosstie or switch tie replacement,
- Surfacing track,
- Combined timbering and surfacing,
- Measurement of track behind surfacing work,
- Rail laying by system gangs,
- Smoothing,
- Cribbing track and spot undercutting,
- Undercutting track out of face,
- Bridge work,
- Laying or transposing welded rail by division maintenance forces, and
- Adjusting welded rail.

Standard Procedure 390 is shown in Figure 1. A discussion of the more important subjects follows. Some of these guidelines are standards within the industry, whereas some are unique to Norfolk Southern.

Track Stability Factors

The procedure starts with several general statements that constitute the theme throughout.

1. Track with CWR must not be disturbed without using the proper slow orders.
2. Track disturbed by new ties, surfacing, or smoothing can lose up to 80 percent of its original resistance to lateral forces.
3. Once disturbed, track stability can only be restored by tonnage at a reduced train speed or by the use of a ballast stabilizer.

Track Conditions

CWR represents a revolutionary advancement in track maintenance by controlling or minimizing the natural expansion of steel caused by temperature increases. This is achieved not by cancelling a physical law, but by preventing rail expansion by using a rigid track structure that is well anchored and embedded in ballast.

Many components make up a track structure, but two of the more important parts in terms of lateral track stability are ballast and rail anchors.

All ballast sections must be maintained to the following minimum standards:

Ballast Location	*Standard*
Jointed rail	
Tangent track	Slopes from ends of top of ties down to roadbed
Curve	
Low side	Same as tangent track
High side	Extends laterally 6 in. from ends of top of ties before sloping down to roadbed
Welded rail	
Tangent track	Extends laterally 6 in. from ends of top of ties before sloping down to roadbed
Curve	
Low side	Same as tangent track
High side	Extends laterally 12 in. from ends of top of ties before sloping down to roadbed

For work that disturbs the track, there are several reminders throughout the procedure that slow orders are not to be removed until a standard ballast section has been restored.

Rail Anchors

Compressive forces are created by the prevention of rail expansion. No part of the track is more important for controlling these forces than the rail anchor.

The point emphasized in Procedure 390 is that all anchors must be applied as required. All missing or defective anchors should be replaced in each timbering cycle. The rail anchors serve no purpose unless they are boxed against the crossties. Therefore, each timbering and surfacing gang is equipped with machines to tighten all anchors against the ties.

The standard pattern of Norfolk Southern is to box anchor every other tie. On curves of 3 degrees and more, every tie is box anchored. All ties are box anchored at ends of trestles and ribbons, and into and away from turnouts.

Track Inspections

Track inspection is the first line of defense for detecting any flaws in the track. During a sudden rise in or extremely high rail temperatures, CWR must be inspected frequently and sometimes daily. This requires some flexibility in work hours and weekend schedules to ensure that employees get time off, while at the same time the needed protection for the safety of train operations is provided.

Some rules and guidelines are as follows:

1. All scheduled track inspections must be maintained.
2. Additional inspections are to be made during sudden changes in temperatures where welded rail or recently disturbed track is subject to misalignment.
3. Weekend inspections are to be made during periods of extreme temperature changes. When a slow order is used for tight track, weekend inspections are necessary.
4. Special attention must be given to track on curves, in dips, at the ends of bridges, and on heavy grades; recently disturbed track; and track worked during the past winter.

Rail temperatures and work situations that disturb the track are key factors in determining when each rule applies. Maintenance personnel must be fully aware of the situations that disturb the track and cause a loss of resistance to lateral forces. Tie renewal, surfacing, and smoothing can create these temporary conditions. When this work is done with changing or high rail temperatures, extreme caution must be taken to prevent track buckling.

Tie Replacement, Surfacing, and Smoothing

Tie renewal, surfacing, and smoothing are each covered separately in the procedure, but because the instructions and guidelines are similar, the three functions are covered together here.

NORFOLK SOUTHERN CORPORATION MW&S STANDARD PROCEDURE	SUPERSEDED DATE 11-01-86 ISSUE DATE 01-01-87	NUMBER 390
TITLE: (6310) MAINTAINING TRACK STABILITY	FILE NUMBER 107-1-829	PAGE 1 of 7

ALL PREVIOUS PROCEDURES AND INSTRUCTIONS IN CONFLICT HEREWITH ARE SUPERSEDED TO THE EXTENT OF THE CONFLICT UPON RECEIPT OF THIS PROCEDURE.

SCOPE AND NATURE

To establish a uniform system for prevention of buckled track due to extreme changes in rail temperature.

OUTLINE OF PROCEDURE

The possibility of track buckling is a constant threat and only alertness, good common sense, and adherence to the following instructions will keep the track in line for the safe operation of the railroad.

PROCEDURE

1. TRACK STABILITY FACTORS ARE:

.01 Track disturbed by surfacing or smoothing can have as little as 20% of the holding power (lateral restraint) of undisturbed track - That is a loss of 80%.

.02 Track Stability, both lateral and vertical, is gained by tonnage over the track or by ballast compaction to a smaller degree.

.03 Track with continuous welded rail must not be disturbed without the proper slow order.

.04 Slow orders must be based on track stability. Stable track is obtained by letting the track settle, under tonnage, at a reduced speed.

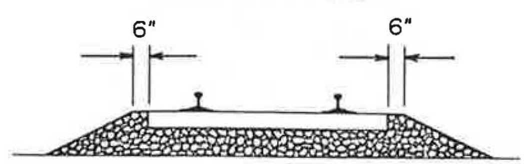
2. TRACK CONDITIONS.

.01 Ballast Sections.

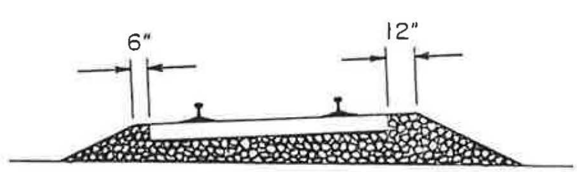
a. A full standard ballast section must be maintained for jointed and welded rail track sections.

b. Standard ballast sections are as shown in the sketches below and on the next page.

WELDED RAIL



TANGENT TRACK



CURVE TRACK

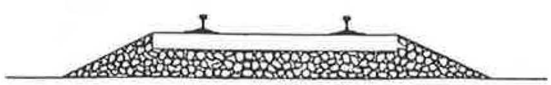
* Denotes revision to procedure last issued 11-01-86.

FORM 11258 (REV. 6/86)

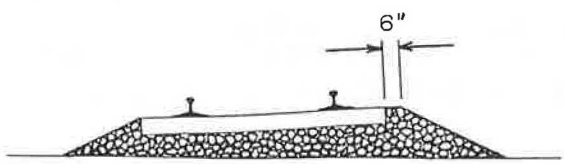
FIGURE 1 Standard Procedure 390, Maintaining Track Stability.

ALL PREVIOUS PROCEDURES AND INSTRUCTIONS IN CONFLICT HEREWITH ARE SUPERSEDED TO THE EXTENT OF THE CONFLICT UPON RECEIPT OF THIS PROCEDURE.

JOINTED RAIL



TANGENT TRACK



CURVE TRACK

.02 Crossties and Switch Ties.

Tie condition should be of sufficient strength to hold gage, surface, and alinement to prevent rail buckling.

.03 Rail Anchors.

a. Rail must be anchored in accordance with applicable procedure(s).

b. In addition to anchors required by above instruction, sufficient anchors must be added to any moving rail which is subject to getting out of line or where anchors do not have sufficient holding power.

.04 Tight Track.

a. Adjustment by cutting may be necessary to welded rail which is tight or not properly adjusted.

b. When track is known to be tight or has moved out of line at the end of a bridge where expansion joints do not exist, it is necessary that rail be cut and adjusted in order to relieve stresses in the track rather than by lining the track.

c. Lining of curves outward may be required for curves which have moved inward due to low temperature from cold weather.

d. Slow Orders must be placed at locations subject to getting out of line until the track condition has been corrected.

3. TRACK INSPECTIONS.

.01 All scheduled track inspections must be maintained.

.02 Additional inspections will be made during sudden changes in temperature where welded rail or recently worked loose track will be subject to getting out of line.

.03 During periods of excessive temperature changes, weekend inspections will be made when required. When a slow order is being run because of tight track, it is necessary to make inspections on Saturday and Sunday.

.04 Special attention must be given to track on curves, in dips, at the ends of bridges, heavy grades, recently disturbed track or track worked during the past winter.

FIGURE 1 *continued*

ALL PREVIOUS PROCEDURES AND INSTRUCTIONS IN CONFLICT HEREWITH ARE SUPERSEDED TO THE EXTENT OF THE CONFLICT UPON RECEIPT OF THIS PROCEDURE.

4. CROSSTIE OR SWITCH TIE REPLACEMENT.

.01 Whenever crossties or switch ties are replaced, a slow order must be used in accordance with instructions below. The foreman or person in charge of the work is responsible for placing the slow order.

a. A 10 m.p.h. slow order must be used in welded and jointed rail territory when the rail temperature is 110°F or above.
b. A slow order of 25 m.p.h., maximum speed may be used when the rail temperature is less than 110°F. Slow orders between 10 and 25 m.p.h. cannot be used on jointed rail.
c. If in doubt as to temperature, follow 110°F or above rail temperature instruction.
d. When a slow order of less than 25 m.p.h. is used, the passage of two tonnage trains is required before slow order is raised.
e. A slow order of 25 m.p.h. maximum speed must be in effect for a sufficient time beyond the work period so that the track will become settled and not be run over by trains at timetable speed immediately after having been disturbed.
f. When the 110°F rail temperature instructions are used, slow orders must remain in effect for at least 2 days of traffic.

.02 Newly installed ties are to be spiked and rail anchors applied in the prescribed spiking and rail anchor pattern at time of installation.

.03 All newly installed ties in welded rail main track must be power tamped before slow order is removed if installed ties exceed two per 39 foot rail.

.04 Upon completion of tie replacement, ballast section must be restored to standard before slow order may be removed.

.05 Removal of Slow Orders.

a. System Gang Work. The System Gang supervisor is responsible for ensuring removal of slow orders unless gang has moved 10 miles or more to a new work location, in which case the track supervisor/roadmaster is responsible for removing the slow order after personal inspection. They must confer with one another to be sure that this is handled properly.
b. Line Maintenance Work. The track supervisor/roadmaster, is responsible for ensuring the removal of the slow order.

5. SURFACING TRACK.

.01 Whenever surfacing work is performed, a slow order must be used in accordance with instructions below. The foreman or person in charge of the work is responsible for placing the slow order.

a. A 10 m.p.h. slow order must be used in welded and jointed rail territory when the rail temperature is 110°F or above.
b. A slow order of 25 m.p.h., maximum speed may be used when the rail temperature is less than 110°F. Slow orders between 10 and 25 m.p.h. cannot be used on jointed rail.
c. If in doubt as to temperature, follow 110°F or above rail temperature instruction.
d. When a slow order of less than 25 m.p.h. is used, the passage of two tonnage trains is required before slow order is raised.
e. A slow order of 25 m.p.h. maximum speed must be in effect for a sufficient time beyond the work period so that the track will become settled and not be run over by trains at timetable speed immediately after having been disturbed.

.02 The runoff made at end of the day must be left in good cross level and alinement with a full standard ballast section, and no condition left which could contribute to buckled track.

FIGURE 1 *continued*

NORFOLK SOUTHERN CORPORATION MW&S STANDARD PROCEDURE	SUPERSEDED DATE 11-01-86 / ISSUE DATE 01-01-87	NUMBER 390
TITLE: (6310T) MAINTAINING TRACK STABILITY	FILE NUMBER 107-1-829	PAGE 4 of 7

ALL PREVIOUS PROCEDURES AND INSTRUCTIONS IN CONFLICT HEREWITH ARE SUPERSEDED TO THE EXTENT OF THE CONFLICT UPON RECEIPT OF THIS PROCEDURE.

.03 If insufficient ballast section exists behind newly surfaced track, the gang supervisor is responsible for placing a proper slow order and advising the track supervisor/roadmaster or division engineer of the condition. The track supervisor/roadmaster or division engineer is responsible for having the ballast section restored and removal of slow order.

.04 Removal of Slow Orders.

a. System Gang Work. The System Gang supervisor is responsible for ensuring removal of slow orders unless gang has moved 10 miles or more to a new work location, in which case the track supervisor/roadmaster is responsible for removing the slow order after personal inspection. They must confer with one another to be sure that this is handled properly.

b. Line Maintenance Work. The track supervisor/roadmaster, is responsible for ensuring the removal of the slow order.

6. COMBINED TIMBERING & SURFACING WORK.

01. Where tie installation is combined with surfacing, i.e., T&S, sections 4 and 5 of these instructions must be applied together. Where instructions combined may conflict, the most restrictive instructions apply and must be followed.

02. In addition, at end of the work week all disturbed track must be fully tamped.

7. MEASUREMENT OF TRACK CONDITIONS BEHIND SURFACING WORK.

.01 Rail Temperature Measurements (System Gangs).

a. Rail temperatures will be taken three times each day and reported to the maintenance of way equipment and material coordinator in Atlanta (Microwave Northern Region 529-2401 or Southern Region 529-1466) along with the daily production report.

b. The temperature will be measured at start of work, middle of day, and at end of work.

c. Rail temperature is measured on the shady side of the web of rail. The thermometer must remain on the rail for at least five minutes and be away from any form of artificial cold or heat other than when rail heater has been used in prescribed manner.

.02 Track Movement Measurements.

a. Where track will be surfaced at a rail temperature of 50°F or below, a Line Maintenance officer will set reference stakes at 3 or more locations on each curve before track is surfaced by T&S or Surfacing Gang.

b. Reference stakes will be set along curves clear of gang activities.

c. A Line Maintenance officer will record the amount of movement one week after each curve is surfaced and furnish the measurements on the prescribed form (see exhibit i) to the office of chief engineer Line Maintenance.

d. The office of chief engineer Line Maintenance will consolidate the reports and furnish summary report to chief engineers Line Maintenance with copies to engineers maintenance of way and division engineers of curve locations where curves moved one inch or more inward at any single point.

e. The division engineer will be held responsible for having curves with average inward movement of one inch or more lined out prior to hot weather, or else track will be slow ordered in hot weather until lining is complete.

8. RAIL LAYING BY SYSTEM GANGS.

.01 When system rail laying schedules are prepared, the chief engineer program maintenance provides a copy of the schedule to the senior chief engineer bridges and structures in order that the Bridge Department can determine required anchoring or use of expansion joints at bridges.

FORM 11258 (REV. 6/86)

FIGURE 1 *continued*

NORFOLK SOUTHERN CORPORATION MW&S STANDARD PROCEDURE	SUPERSEDED DATE 11-01-86 ISSUE DATE 01-01-87	NUMBER 390
TITLE: (6310T) MAINTAINING TRACK STABILITY	FILE NUMBER 107-1-829	PAGE 5 of 7

ALL PREVIOUS PROCEDURES AND INSTRUCTIONS IN CONFLICT HEREWITH ARE SUPERSEDED TO THE EXTENT OF THE CONFLICT UPON RECEIPT OF THIS PROCEDURE.

.02 Whenever rail is laid in tracks with a timetable speed greater than 25 m.p.h., a slow order must be used. The system gang supervisor is responsible for ensuring the placement of the slow order.

a. Recommended maximum speeds for slow orders when laying rail is 25 m.p.h.
b. Dependent upon other track conditions (such as alinement, tie condition, surface, or rail condition) a speed less than 25 m.p.h. may be required.

.03 Before slow orders can be raised:

a. All joints tightly bolted with at least two bolts each rail end,
b. rail spiked in the prescribed pattern,
c. rail anchors must be installed tight against the ties in prescribed pattern,
d. all down ties fully tamped, and
e. standard shoulder ballast section must be provided.

.04 Where speed has been restricted to less than 25 m.p.h. for rail laying, the passage of one tonnage trains is required before raising the speed to 25 m.p.h. or greater

.05 The division engineer or the track supervisor/roadmaster after personal inspection of the rail laid will determine the appropriate speed to run on the track.

.06 If the rail temperature is below 80°F, rail heater must be used to raise the rail temperature ahead of spiking to a temperature of 85°F to 100°F, ideally 95°F.

.07 Throughout welded rail laying, slack must be removed by use of rail pulling equipment.

.08 The rail gang supervisor is responsible for ensuring that the rail temperature be taken at time of anchoring for each strand (single gang) or each ribbon (dual gang) and reporting to the maintenance of way equipment and material coordinator in Atlanta (microwave Northern Region 529-2401 or Southern Region 529-1466) along with the daily production report. These temperatures will in turn be furnished to the office of chief engineer Line Maintenance.

.09 The office of chief engineer Line Maintenance will prepare/update rail temperature charts and furnish to the chief engineers Line Maintenance, engineers maintenance of way, and the division engineers for their territory.

.10 The division engineers must review the rail temperature of all welded rail laid on his territory and make adjustments where required.

9. SMOOTHING.

.01 Good judgement should be exercised in smoothing during hot weather and extreme temperature changes.

.02 Welded rail should not be smoothed when rail temperature is above 110°F unless such smoothing is necessary to afford safe passage of trains.

.03 Slow Orders.

a. A 10 m.p.h. slow order must be placed at any location in jointed or welded rail territory when it is necessary to smooth track and the rail temperature is 110°F or above.
b. A slow order of 25 m.p.h. maximum speed may be used when track is smoothed at a rail temperature of less than 110°F. Slow orders between 10 and 25 m.p.h. cannot be placed on jointed rail.
c. If there is a possibility that rail temperature will rise to 110°F later in the day, a 10 m.p.h. slow order must be used until track has settled under traffic and is safe for timetable speed.
d. A slow order of 25 m.p.h. maximum speed must be in effect for a sufficient time beyond the work period so that the track will become settled and not be run over by trains at timetable speed immediately after having been disturbed.
e. The track supervisor/roadmaster, assistant track supervisor/assistant roadmaster, or the foreman in charge of the work is responsible for placing and removing the slow order.
f. If more than 4 continuous ties are hand tamped in welded rail territory, a 25 m.p.h. slow order must be in effect until track is power tamped and track is settled for timetable speed.

FORM 11258 (REV. 6/86)

FIGURE 1 *continued*

ALL PREVIOUS PROCEDURES AND INSTRUCTIONS IN CONFLICT HEREWITH ARE SUPERSEDED TO THE EXTENT OF THE CONFLICT UPON RECEIPT OF THIS PROCEDURE.

.04 When smoothing or restoring prescribed elevation in curves, each tie must be fully tamped under each rail to eliminate voids between tie and ballast section.

.05 Tie cribs must be filled with ballast at any point disturbed by smoothing and track left in good alinement.

.06 When Line Maintenance smoothing gangs are performing any surfacing work, they will also be governed by the instructions under "Surfacing Track" in 5.01 d on page 3.

10. CRIBBING TRACK AND SPOT UNDERCUTTING.

.01 A 25 m.p.h. maximum speed slow order must be used when cribbing tracks of foul ballast, cribbing road crossings, and spot undercutting.

.02 After a full standard ballast section has been restored, a slow order of 25 m.p.h. maximum speed must be in effect for a sufficient time beyond the work period so that the track will become settled and not be run over by trains at timetable speed immediately after having been disturbed.

11. UNDERCUTTING TRACK OUT OF FACE.

.01 The track supervisor/roadmaster, division engineer or an officer designated by the division engineer must be with any track undercutting operation and is responsible for ensuring placement and removal of slow order.

a. Following the undercutting operation, a slow order of 10 m.p.h. must be used and must remain for a minimum of 24 hours.

b. After 24 hours, speed may be increased to a maximum of 25 m.p.h. (Jointed rail may not have a slow order between 10 and 25 m.p.h.) The 25 m.p.h. slow order must remain in effect as follows:

Annual Tonnage	Time, at least
Less than 10 million	4 days of traffic
10 million or greater	2 days of traffic

c. Tangent track that cannot be restored to proper alinement during the heat of the day (noon to 6 p.m.) account tight track must be cut and adjusted in accordance with applicable procedure before slow order is raised or removed.

.02 Measurements of track movement on curves behind surfacing work done in conjunction with undercutting operation will be as covered in section 7 with the following exceptions:

a. Measurements will be made on curves if rail temperature is 70°F or less when track is undercut.

b. Stakes will be set clear of all work activities and initial measurements made before track is undercut.

*12. BRIDGE WORK.

* .01 A slow order will be used when bridges ties are installed.

* .02 Renewing Bridge Ties On Open Deck Bridges in Welded Rail.

a. When the rail temperature is in the range of 10°F below the laying or adjusted temperature up to 110°F, not more than ten consecutive ties may be unspiked at one time, and then only when adjacent ties are secured in place with drift or hook bolts, AND rails are strutted apart with substantial timber and bound together tightly with load binder or come-along.

b. When welded rail temperature is above 110°F and the rails are bound as required in sub-paragraph a above, not more than five consecutive ties may be unspiked at one time.

* .03 When jointed rail is extremely tight due to hot weather conditions, it should be handled as welded rail.

* .04 When renewing ties on ballast deck bridges, the instructions in section 4 (Crosstie or Switch Tie Replacement) governs.

* .05 When a B&B gang raises or disturbs the track approach to an open deck bridge, all items under section 9 (Smoothing) must be observed by the B&B forces.

* Denotes revision to procedure last issued 11-01-86.

FIGURE 1 *continued*

ALL PREVIOUS PROCEDURES AND INSTRUCTIONS IN CONFLICT HEREWITH ARE SUPERSEDED TO THE EXTENT OF THE CONFLICT UPON RECEIPT OF THIS PROCEDURE.

.06 When track is known to be tight or has moved out of line at the end of a bridge where expansion joints do not exist, it is necessary that the rail be cut and adjusted in accordance with applicable procedure in order to relieve stresses in the track rather than by lining.

.07 When ties are renewed or track is otherwise disturbed across a bridge or within 200 feet of a bridge, special attention is required to ensure that rail anchors are installed in accordance with standards and expansion joints, where used, are in proper condition before temporary speed restrictions are removed.

13. LAYING OR TRANSPOSING WELDED RAIL BY LINE MAINTENANCE.

.01 Whenever rail is to be laid across bridges, the division engineer is to notify the B&B supervisor well in advance of laying so that the Bridge Department can determine required anchoring or use of expansion joints.

.02 The existing applicable procedures are to be followed when rail is laid or transposed by Line Maintenance forces. It is imperative that the reporting be made in accordance with exhibit ii.

.03 The track supervisor/roadmaster or an officer designated by the division engineer must be with any Line Maintenance forces transposing or laying welded rail.

.04 Transposing or replacement of curve worn rail shall be performed between May 15th and September 15th where possible.

.05 When welded rail is laid, rail must be anchored at a rail temperature of 75°F or greater.

.06 When a rail heater is used the rail will be heated to a rail temperature between 85°F and 100°F, ideally 95°F, ahead of the spiking operation.

.07 The division engineer must review the rail temperature of all welded rail laid on his territory and make adjustments where required.

14. ADJUSTING WELDED RAIL.

.01 The existing applicable procedures are to be followed when welded rail is adjusted. It is imperative that the required reporting be made in accordance with exhibit iii.

.02 Rail Adjustment by Tie, Surfacing, or T&S Gangs.

a. Track being worked by T&S, Tie, or Surfacing Gangs may require that the rail be adjusted immediately to maintain proper alignment of track.
b. Since the rail is in compression, it must be cut with a torch, realigned, holes drilled, and angle bars applied.
c. Each T&S, Tie, and Surfacing Gang is required to have available:

(1) A rail drill with proper size bits.
(2) Two pair of angle bars of same weight as rail being worked, with necessary bolts and nutlocks.

d. When System Gangs have made emergency rail adjustments, they must notify Line Maintenance immediately so that Line Maintenance forces can complete adjustment of rail in accordance with applicable procedures.

.03 Rail Adjustment by Line Maintenance. The track supervisor/roadmaster or individual designated by the division engineer must be with any Line Maintenance forces adjusting welded rail.

APPROVED:

Assistant Vice President-Maintenance

FORM 11258 (REV. 6/86)

FIGURE 1 ***continued***

When crossties or switch ties are replaced or surfacing and smoothing is performed, a slow order must be used as follows:

1. A 10-mph slow order must be used in welded- and jointed-rail territory when the rail temperature is 110°F or above.
2. A slow order not to exceed 25 mph should be used when the rail temperature is less than 110°F.
3. If the exact temperature is not known, the instructions for rail 110°F or above should be followed.
4. When a slow order of less than 25 mph is used, the passage of two tonnage trains is required before the order can be lifted.
5. A slow order of 25 mph must be in effect for a sufficient length of time after work is performed so that the disturbed track becomes settled before trains are run over it at timetable speed.
6. When the 110°F rail temperature instructions are used, slow orders must remain in effect for at least 2 days of traffic.
7. For smoothing, if more than four continuous ties are hand tamped in welded-rail territory, a 25-mph slow order must be in effect until the track is power tamped and has settled.
8. Upon completion of work, the ballast section must be restored to standard condition before the slow order may be removed.

Because rail temperature is critical to lateral stability, it is the motivating factor for the formulation of many of Norfolk Southern's guidelines. All production gangs are required to measure rail temperatures at least three times daily. These temperatures are reported along with the production reports to the Atlanta office.

In the early 1970s several buckled-track derailments occurred on curves that had been surfaced the previous winter. Because the track had been worked below the rail-laying temperature, the disturbed track moved inward during the work cycle. No record was made of this movement, and therefore no adjustment was made to the rail. As a result, each spring and summer track alignment problems occurred. To prevent these problems, instructions were written to measure movement of curves that are disturbed during cold weather.

Measurement of Track Conditions Behind Surfacing Work

When track is to be surfaced at a rail temperature of 50°F or below, reference stakes are to be set on curves and measured ahead of the work.

One week after the production gang performs its work, measurements are to be taken to record any movement of the curve. This information is furnished to the chief engineer's office where a report is prepared listing all locations that moved inward 1 in. or more. This information is sent to the division engineer, who is responsible for adjusting rail on all curves that moved 1 in. or more inward, on average.

It is not always possible to work track at or above the rail-laying temperature, but the practice of measuring curves for inward movement has prevented many problems.

Rail Laying

There is no substitute for a good rail-laying job to prevent lateral track stability problems. A number of quality control measures must be performed correctly to achieve stability, such as line, gage, application of all fasteners, plates, spikes, and others. Each is covered in the procedure; only a few of the instructions relative to the establishment of the rail-laying temperatures are mentioned here.

1. If rail temperatures are below 80°F, a rail heater must be used. Rail must be heated so that the temperature at the time of spiking and anchoring is 85° to 100°F, ideally 95°F.
2. Throughout welded rail laying, slack created by the rail heater and the laying process must be continuously removed by use of rail-pulling equipment.
3. The rail gang supervisor is responsible for taking the rail temperature for each ribbon just before the anchoring process.
4. Temperature charts of all rail-laying jobs are furnished to the division engineers, who must review the charts and make rail adjustments where required.

Other subjects, such as cribbing, undercutting, bridge work, transposing of rail, and adjustment of rail are covered in the procedure. These are all critical components of rail laying and are covered in some detail in Figure 1.

Train Handling over Welded Rail

Some people in the industry and some researchers contend that procedures should also be issued for train handling over welded rail to improve track stability. Although train operations can create conditions that may cause track alignment problems, Norfolk Southern has not issued guidelines for train operation. Problems usually occur at ends of bridges, on heavy grades, in dips, and on curves. For this reason special attention must be given to these locations during track inspections for any telltale signs of problems.

Although train handling itself is not covered in Norfolk Southern procedures, adequate protection against poor train handling over disturbed track is provided by slow-order procedures and track inspection requirements for critical locations.

This procedure, written with field personnel in mind, has been a significant factor in the prevention of buckled track on the Norfolk Southern rail system. Personnel have been given clear, precise guidelines to follow to avoid problems in situations that most likely will lead to unstable track conditions. This procedure was distributed to all field personnel in a pocket-size 3- by 5-in booklet. This was done so that the procedure would be in their possession at all times, in the field where needed, not in a standard procedure three-ring notebook back at the office.

Procedures and standards are an absolute necessity for a safe, uniform system of laying and maintaining CWR. However, the standards are effective only if they are properly communicated to and understood by all field personnel who actually perform the work.

Training Programs

Two steps are performed to communicate the procedures to the field personnel:

First, in the spring of each year, staff meetings are scheduled at several central points throughout the system. These meetings are conducted by the assistant vice president of maintenance and the chief engineers. The theme of the meetings is the prevention of buckled track. The discussions are primarily for the first-level supervisory officers, the field personnel. The reasons why sun kinks and buckled track occur are explained, and Standard Procedure 390 is reviewed section by section. These meetings are mandatory for all maintenance-of-way officers and have been part of the training program since 1974.

The second step is for the division engineers to take the message back to the field and review the instructions with the foremen.

This procedure is conducted annually. Some may ask, "Is it all really necessary?" Working with CWR must be given top priority for safety of operations, and this is one method of driving the point home to those actually involved in the day-to-day field work. After the inception of this program in 1974, the number of buckled-track incidents dropped dramatically.

These instructions are constantly reviewed and evaluated for effectiveness. After a recent review, the following training programs for field personnel were added.

1. All scheduled employees promoted to field track or bridge supervisory positions are given 2 weeks of classroom training with on-track instructions.
2. All officers and some scheduled track employees take a written exam on FRA track safety standards as part of the annual spring meetings.
3. Foremen and assistant foremen attend a formal training school consisting of 2 weeks of classroom work with on-track instruction.

These programs cover all phases of track maintenance, including Standard Procedure 390, and should improve the effectiveness of maintenance practices.

CONCLUSION

Have the procedures, guidelines, and training programs been effective? Employees of Norfolk Southern think so. Over the last 10 years the railroad has had 13 derailments caused by

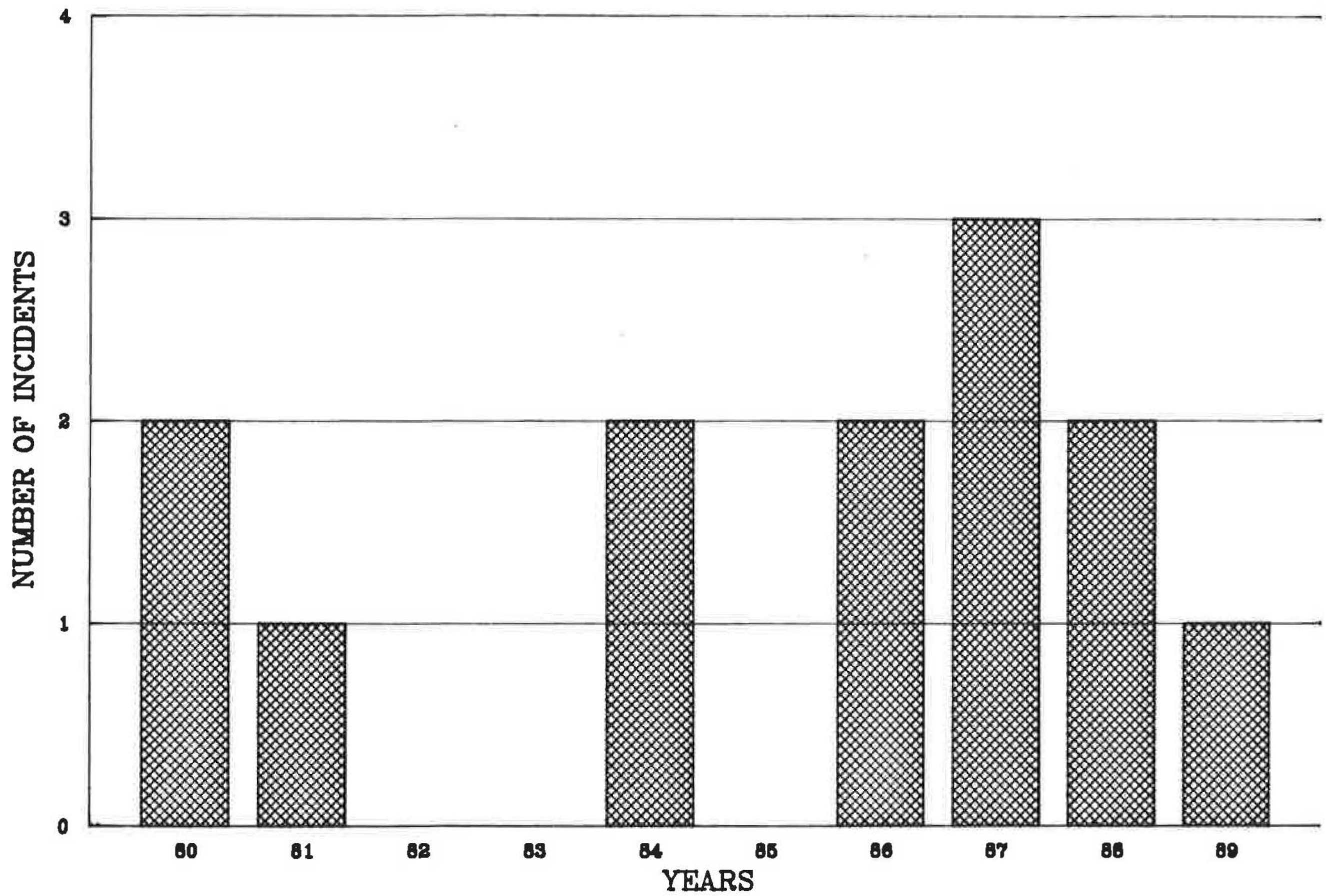


FIGURE 2 Norfolk Southern Corporation buckled-track derailments.

buckled track. No derailments occurred in 1982, 1983, and 1985 (Figure 2). Several of the derailments involved only one car.

In conclusion, it is believed that Norfolk Southern has a good program of instructions and guidelines for working with CWR. These instructions are based on sound engineering decisions for the conditions encountered on the rail system. To make this program effective, employee training is provided annually at the field level. Employees are committed to safety of operations, and it is believed that welded rail can be worked with safety under any circumstances if personnel are constantly alert to the conditions that can cause buckled track and follow the procedures for maintaining track stability.

A statement distributed at the annual spring meetings sums up Norfolk Southern's philosophy: Disturbed track in hot weather plus failure to follow instructions equals buckled track.