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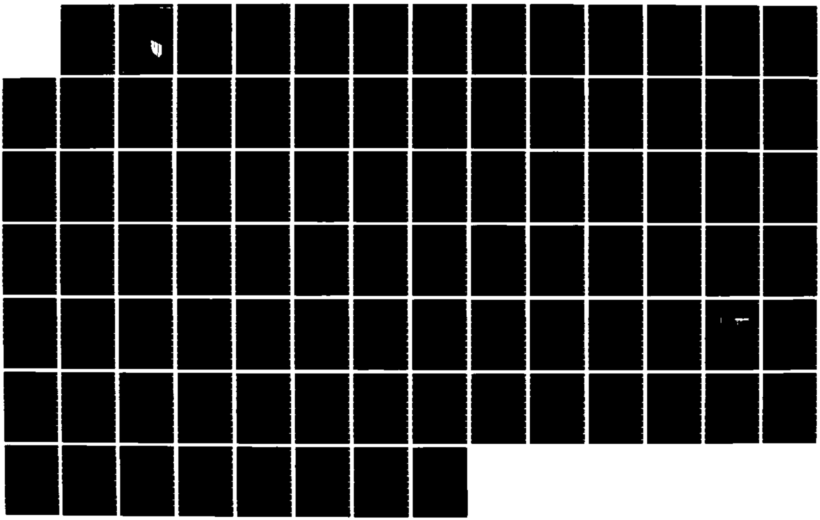

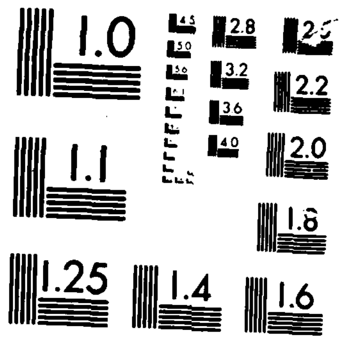


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US Army Corps
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Construction Engineering
Research Laboratory

USA-CERL

TECHNICAL REPORT M-86/01
May 1986
The Railroad Maintenance Management System

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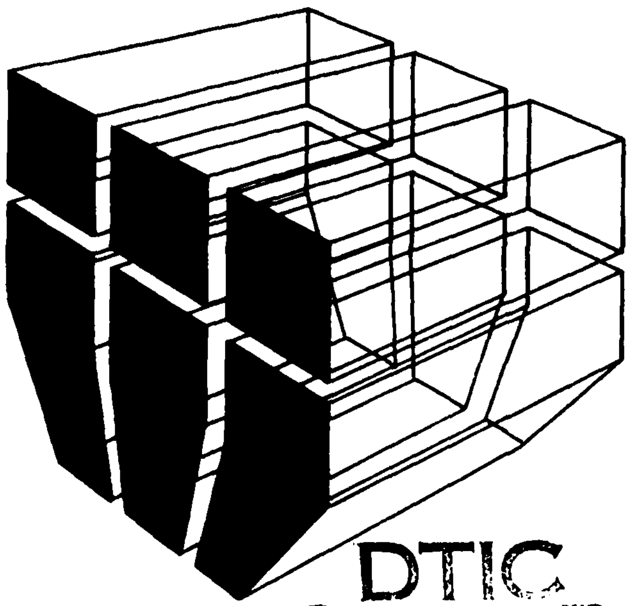
# Development of the U.S. Army Railroad Track Maintenance Management System (RAILER)

by
M. Y. Shahin



U.S. Army Facilities Engineers are responsible for maintaining more than 3000 miles of railroad track. The track is dispersed in small lots and is analogous to industrial rather than commercial trackage. There is currently no standard method for gathering track inventory and condition data and no standard method of determining the track's condition. This report presents an overview of the proposed U.S. Army Railroad Maintenance Management System (RAILER) and recommends procedures for track evaluation.

RAILER will consist of subsystems for network definition, data collection, data storage and retrieval, network data analysis, and project data analysis. Subsystem development will be highly dependent on track condition evaluation procedures. Two major evaluation categories have been identified: track structural condition and track operational condition.





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U.S. Army Facilities Engineers are responsible for maintaining more than 3000 miles of railroad track. The track is dispersed in small lots and is analogous to industrial rather than commercial trackage. There is currently no standard method for gathering track inventory and condition data and no standard method of determining the track's condition. This report presents an overview of the proposed U.S. Army Railroad Maintenance Management System (RAILER) and recommends procedures for track evaluation.

RAILER will consist of subsystems for network definition, data collection, data

BLOCK 20 (Cont'd)

storage and retrieval, network data analysis, and project data analysis. Subsystem development will be highly dependent on track condition evaluation procedures. Two major evaluation categories have been identified: track structural condition and track operational condition.

## FOREWORD

This research was performed for the Office of the Assistant Chief of Engineers (OACE) under Project 4A162731AT41, "Military Facilities Engineering Technology"; Technical Area C, "Operations, Maintenance, and Repair"; Work Unit 042, "The Railroad Maintenance Management System." The work was performed by the Engineering and Materials Division (EM), U.S. Army Construction Engineering Research Laboratory (USA-CERL). The OACE Technical Monitor was Mr. Robert Williams, DAEN-ZCF-B.

The assistance of the following people is gratefully acknowledged: Mr. Don Plotkin (USA-CERL); Mr. Mike Flaherty (Army Material Command); and the pavement and railroad engineers of the Army Major Commands.

Dr. Robert Quattrone is Chief of USA-CERL-EM. COL Paul J. Theuer is Commander and Director of USA-CERL, and Dr. L. R. Shaffer is Technical Director.

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## DEVELOPMENT OF THE U.S. ARMY RAILROAD TRACK MAINTENANCE MANAGEMENT SYSTEM (RAILER)

## 1 INTRODUCTION

### Background

The U.S. Army's Facilities Engineers (FEs) are responsible for maintaining more than 3000 miles* of railroad track. This track is dispersed in small lots and is analogous to industrial rather than commercial trackage. Since funding for track maintenance has low priority, much of the needed maintenance and repair (M&R) has been deferred. If this trend continues, some of the track may deteriorate to a point where it can no longer support its mobilization mission. Another major problem of keeping the Army's railroad track in good repair has been that installations do not have personnel who are knowledgeable about track maintenance. This, coupled with a lack of specific guidance that can be applied consistently among installations, has caused track repair to be both inadequate and expensive.

There is currently no standard method for gathering track inventory and condition data or for determining the track's condition. Therefore, the U.S. Army Construction Engineering Research Laboratory (USA-CERL) performed extensive research to define the Army's track maintenance problems and available maintenance management systems, and to recommend research products that would alleviate the problems. U.S. Army Major Command (MACOM) engineers, Strategic Mobility personnel, and track maintenance personnel were interviewed to obtain input about Army track maintenance problems. In addition, 27 large operating railroad firms, 14 firms operating shortline railroad tracks, the Federal Railroad Administration (FRA), and private railroad consultants were surveyed to determine what system, if any, they used to manage their track maintenance operations. Results of the surveys and interviews, documented in a USA-CERL Technical Report[1], showed that there is no complete track maintenance management system that can be readily adapted to Army use. It was determined that the most efficient way of providing a track maintenance management system would be to design one specifically for the Army operations.

USA-CERL has developed and successfully used PAVER--a computerized management system for pavement maintenance.[2] It was decided that the generic concepts of maintenance management developed for PAVER could be adapted to a

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*Metric conversion factors are provided on p 24.

[1] S. C. Solverson, M. Y. Shahin, and D. R. Burns, *Development of a Railroad Track Maintenance Management System for Army Installations: Initial Decision Report*, Technical Report M-85/04/A149491 (U.S. Army Construction Engineering Research Laboratory [USA-CERL] 1984).

[2] M. Y. Shahin and S. D. Kohn, *Pavement Maintenance Management for Roads and Parking Lots*, Technical Report M-294/ADA110296 (USA-CERL and U.S. Air Force Engineering and Services Center, October 1981); M. Y. Shahin and S. D. Kohn, *Overview of the PAVER Pavement Management System and Economic Analysis of Field Implementing the PAVER Management System*, Technical Manuscript M-310/ADA116311 (USA-CERL, 1982).

railroad track maintenance management system; however, special attention would have to be given to the technological differences between pavements and railroads.

## Purpose

The purpose of this report is to describe the components and recommended evaluation procedures for the proposed U.S. Army Railroad Track Maintenance Management System (RAILER).

## Approach

RAILER subsystems were developed that would provide the basis for collecting, storing, and analyzing data on Army railroad track. Data were gathered on various methods for evaluating railroad track. Based on this information, recommendations were made regarding the best methods for evaluating track with the proposed RAILER system.

## Mode of Technology Transfer

The recommended procedures presented in this report are currently being developed for implementation in both manual and automated form. It is recommended that these procedures be documented in an Army Technical Manual.

## 2 OVERVIEW OF THE RAILER SYSTEM COMPONENTS

The basic subsystems of any facility maintenance management system (e.g., for a railroad) consist of network definition, data collection (including condition survey), data storage and retrieval, network data analysis, and project data analysis. Figure 1 shows the relation between these subsystems. The development of each subsystem for a given facility should be technologically based rather than blindly adapted from another facility's management system. Following is a brief description of each subsystem proposed for the RAILER system.

### Network Definition

A track network may be defined in terms of mileposts, switch locations, grade crossings, and structures such as bridges. The network should be divided into uniform sections that are similar in construction and condition and that are subjected to the same traffic loadings. These sections represent the smallest management units for assessing major rehabilitation needs.

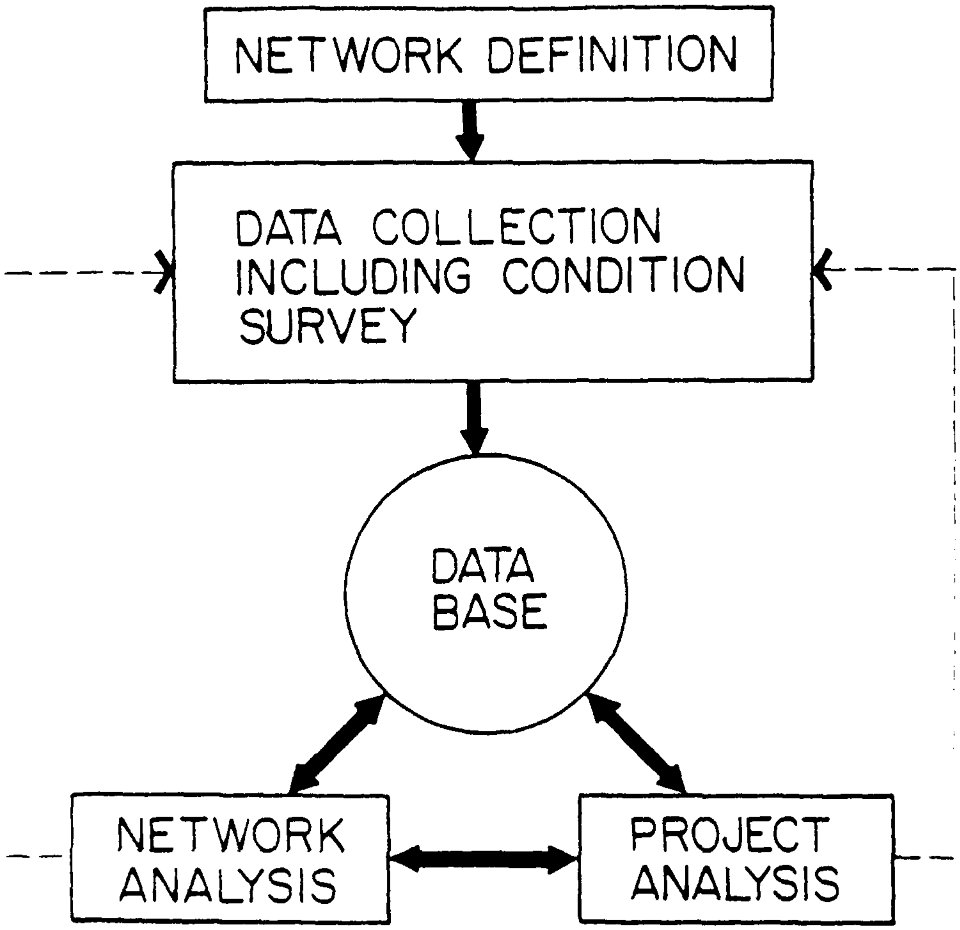


Figure 1. Generic facility maintenance management system.

### Data Collection

Data collection refers to conducting a physical inventory of both the track structure and its components' condition through a condition survey. Data should also be gathered on the traffic that uses the track, including the load intensity and the number of load repetitions. Data collection should be limited to the amount required for project- and network-level analyses (defined below). Exceeding these requirements will not be cost-effective and could cause the entire management system to fail because the system will become cumbersome and require too much data collection effort.

### Database

A database can be manual (file cabinet) or automated (computer). A computerized system is preferred because of its convenience, smaller cost, and greater expediency. An inefficient or badly designed database will cause the overall system to be inefficient. The objective of the database is to provide an expedient, user-friendly means of data storage and retrieval. In the past 2 years, many "database manager" software packages have been developed for microcomputers with features that were formerly available only on large-frame computers. Some of these packages offer excellent support for screen-formatted data entry, report generation using conversational language, and interface of engineering analysis programs with the database.

### Network Analysis

Development of the network analysis programs is difficult, requiring the cooperation and involvement of the systems' ultimate users. Network analysis includes budget planning, budget optimization, project identification and prioritization, and network inspection scheduling. To avoid duplication of efforts, it is best to coordinate network inspection with the maintenance standards inspection.

### Project Analysis

Project-level analysis determines the best track rehabilitation alternative and requires more detailed condition data than network analysis. One major factor in selecting the best rehabiltation alternative is life-cycle costing. Emphasis should be placed not only on initial rehabilitation cost, but also on an alternative's future maintenance costs. The economic analysis developed as part of the PAVER system[3] can be used for RAILER; however, guidelines for providing track information inputs to the analysis procedure must first be developed.

Subsystem development is highly dependent on the track condition evaluation procedures used. These procedures are discussed in Chapter 3.

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[3]M. Y. Shahin and S. D. Kohn, *Pavement Maintenance Management for Roads and Parking Lots*; M. Y. Shahin and S. D. Kohn, *Overview of the PAVER Pavement Management System* and *Economic Analysis of Field Implementing the PAVER Management System*.

# 3 RAILROAD TRACK CONDITION EVALUATION CONCEPTS AND RECOMMENDED EVALUATION PROCEDURES

The railroad track and its support system have four main components: the rail and ties, which make up the basic track structure, and the ballast and subgrade, which make up the foundation. Besides providing direct support for train traffic loads, each component distributes wheel loads over an increasingly larger area, thus minimizing pressure on the subgrade. For the track system (track and foundation) to withstand the loads imposed by train traffic, each component must have enough structural integrity to carry out its dual role of load support and load distribution.

Besides providing structural support, the track system must also maintain track geometry--the proper position and alignment of two rails. A deterioration of either the track's strength or geometry can make it unsuitable for service.

There are two major track condition categories: structural condition and operational condition. Structural condition is a measure of the load-carrying capacity of the track structure. It accounts for both the magnitude of the wheel loads and the number of load repetitions the track system can handle before failure occurs. Structural condition is evaluated using a track modeling technique and knowledge of the strength of the track system's individual components, including rail, ties, ballast, and subgrade. Operational condition is a measure of M&R needs, as well as of the track system's safety. Operational condition is evaluated based on the condition of the individual components of the track structure as well as the geometric condition of the track. The condition of each track component is evaluated as follows:

1. Rail condition
   a. Internal defects such as cracks
   b. External defects such as wear

2. Tie condition
   a. Number of defective ties
   b. Severity of defects
   c. Arrangement of defective ties

3. Ballast and subgrade
   a. Degree of fouling and degradation
   b. Drainage condition

4. Track geometry condition
   a. Gauge
   b. Crosslevel
   c. Profile
   d. Alignment.

There is currently no standard method for evaluating track conditions as a whole--i.e., one that considers both track structural and operational condition.

Before recommending the track evaluation procedures to be used with RAILER, it was necessary to gather information on currently used concepts and procedures. Three

railroad engineering consultants were contracted to perform preliminary studies and provide necessary background in the following areas:

1. Ballast and subgrade evaluation and overall track strength condition evaluation (Appendix A)

2. Track geometry evaluation (Appendix B)

3. Tie condition evaluation (Appendix C).

Meetings were also held with the U.S. Army Pavement and Railroad Maintenance Committee, which includes Army MACOM railroad engineers. During these meetings, various track condition evaluation concepts were presented and critiqued.

The recommendations presented in this paper are based on the consultants' reports, authors' views, and input from the Army committee. The information obtained identified two major evaluation categories: track structural condition and track operational condition. The following sections present recommendations for evaluating each of these categories.

## Structural Condition Evaluation

Two evaluation procedures are recommended. The first one is approximate, but is simple to use. The other method provides in-depth analysis as a basis for determining cost-effective M&R alternatives. In principle, both procedures are based on mechanistic analysis of track behavior, and on relating that behavior to track performance. However, the inputs for the approximate procedure do not have to be based on direct measurements of material properties.

The overall structural evaluation of the track is a function of its components, which include subgrade, ballast, tie, and rail, and of the load to which the track is subjected. The effect of each component on track structural condition indicators was studied in cooperation with Professor Marshall Thompson of the University of Illinois, using the ILLI-TRACK computer system.[4] Table 1 presents typical results obtained using ILLI-TRACK. Using subgrade strength, ballast thickness, tie spacing, rail size, and load as inputs to ILLI-TRACK, one can determine the following track structural condition indicators:

1. Tie reaction in kips (ballast bearing pressure can be computerized as tie reaction divided by tie width times effective length)

2. Tie deflection

3. Subgrade stress ratio

4. Rail bending stress.

Each indicator can be used to determine the track's adequacy or inadequacy to carry a specific load for a given number of repetitions.

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[4] S. D. Tayabji and M. R. Thompson, *Program ILLI-Track—A Finite Element Analysis of Conventional Railway Support System—User's Manual and Program Listing*, Ballast and Foundation Materials Research Program, Department of Civil Engineering, University of Illinois (Federal Railroad Administration [FRA], Report No. FRA-OR&D-76-257).

## Table 1

### ILLI-TRACK Response Summary

| Ballast thickness in. | Subgrade* | Max. tie reaction, kips | Tie Δ, mils | Subgrade stress, psi $\sigma_1$ | $\sigma_3$ | $\sigma_D$ | Relative subgrade σ, %** |
|---|---|---|---|---|---|---|---|
| 12 | Medium | 18.5 | 153 | 26.7 | 16.9 | 9.8 | 43 |
| 18 | Medium | 20.7 | 138 | 24.3 | 15.4 | 8.9 | 39 |
| 24 | Medium | 22.4 | 133 | 22.1 | 14.2 | 7.9 | 34 |
| 12 | Soft | 16.6 | 299 | 23.6 | 16.6 | 7.0 | 54 |
| 18 | Soft | 19.3 | 266 | 21.7 | 15.3 | 6.4 | 49 |
| 24 | Soft | 21.5 | 248 | 20.0 | 14.2 | 5.8 | 45 |
| 12*** | Very Soft | 15.4 | 493 | ---- | ---- | ---- | 100 |
| 18 | Very Soft | 17.4 | 479 | 17.1 | 13.0 | 4.1 | 66 |
| 24 | Very Soft | 20.2 | 438 | 15.6 | 11.9 | 3.7 | 60 |
| 6 | Medium | 16.7 | 159 | 30.0 | 18.8 | 11.2 | 49 |
| 6 | Soft | 15.1 | 319 | 26.0 | 18.4 | 7.6 | 59 |
| 6 | Very Soft | 18.2 | 553 | 34.2 | 28.0 | 6.2 | 100 |

*Subgrade strengths: medium - $q_u$ = 23 psi; soft - $q_u$ = 13 psi; very soft - $q_u$ = 6.2 psi.

**Relative subgrade stress = 100 (subgrade stress/subgrade strength) = 100 ($\sigma_D/q_u$).

***Extensive subgrade and ballast failure; stress data are not valid.

For the approximate procedure, a parameter study using ILLI-TRACK (or a similar mechanistic model) is recommended from which a nomograph, such as that shown in Figure 2, can be constructed. A methodology must be developed for obtaining inputs without having to use sophisticated testing or analysis. If the track strength is de'ermined to be inadequate or questionable, then the detailed structural evaluation can be requested.

For the in-depth evaluation procedure, similar nomographs can be developed, but the input would require more direct measurements, and the output would be in terms of allowable specific load value and an associated number of repetitions. An alternate method would be the direct use of the selected mechanistic model on a project-by-project basis.

The study provided in Appendix A was performed using ILLI-TRACK to illustrate the relative effect of rail size and tie spacing on the structural strength indicators. Table 2 and Figures 3 through 6 present the study results. It should be noted that a tie spacing of 40 in. was used to simulate a case in which every other tie is bad. However, future parameter studies should consider various arrangements of bad ties; from this limited study, the importance of both tie condition and rail size cannot be overemphasized. The study was performed assuming a soft subgrade. Table 1 demonstrates the significance of the subgrade class.

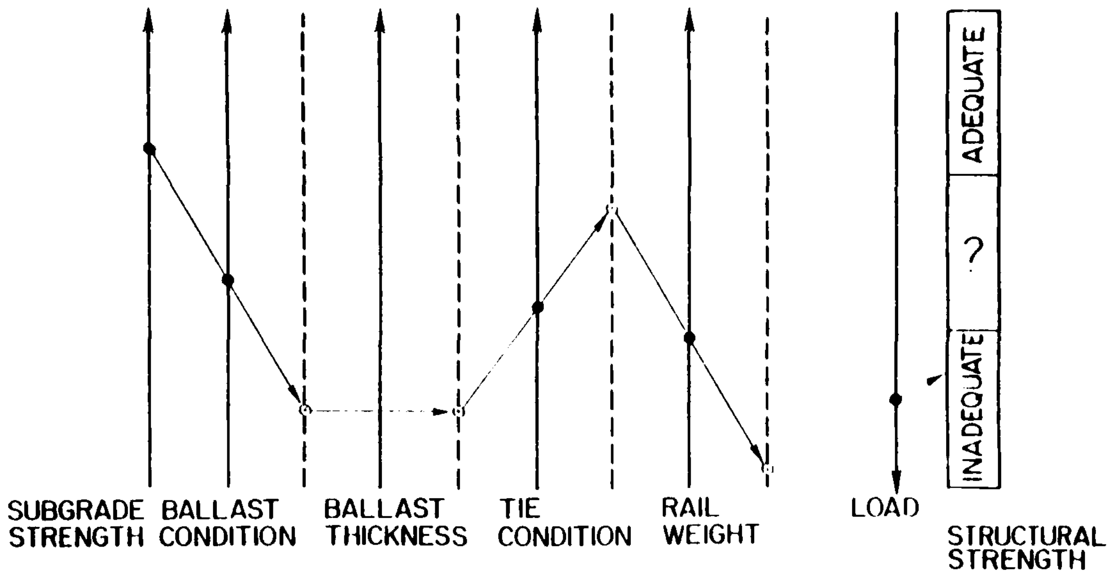



**Figure 2.** **Conceptual nomograph recommended for U.S. Army railroad track structural evaluation.**

## Table 2

## ILLI-TRACK Comparisons

| Rail size | Tie spacing in. | Max. tie reaction, kips | Tie Δ, mils | Subgrade stress, psi $\sigma_1$ | $\sigma_2$ | $\sigma_D$ | Relative subgrade σ, % | Rail bending stress, ksi |
|---|---|---|---|---|---|---|---|---|
| 132# | 20 | 16.6 | 299 | 23.6 | 10.0 | 7.0 | 54 | 19.3 |
| 132# | 40 | 30.3 | 456 | 25.8 | 16.4 | 9.4 | 72 | 22.1 |
| 90# | 20 | 19.5 | 310 | 24.4 | 16.6 | 7.9 | 61 | 28.9 |
| 90# | 40 | 31.7 | 459 | 26.4 | 16.3 | 10.1 | 78 | 32.8 |
| 60# | 20 | 26.0 | 311 | 26.3 | 16.0 | 9.5 | 73 | 43.0 |

*Relative subgrade stress = 100 (subgrade stress/subgrade strength) = 100 ($\sigma_D/q_u$);
Subgrade strength: soft $q_u$ = 13 psi.

NOTES: 1. Ballast thickness - 12 in. of area #4
2. Soft subgrade
3. Tie size: 9 in. width x 7 in. thick

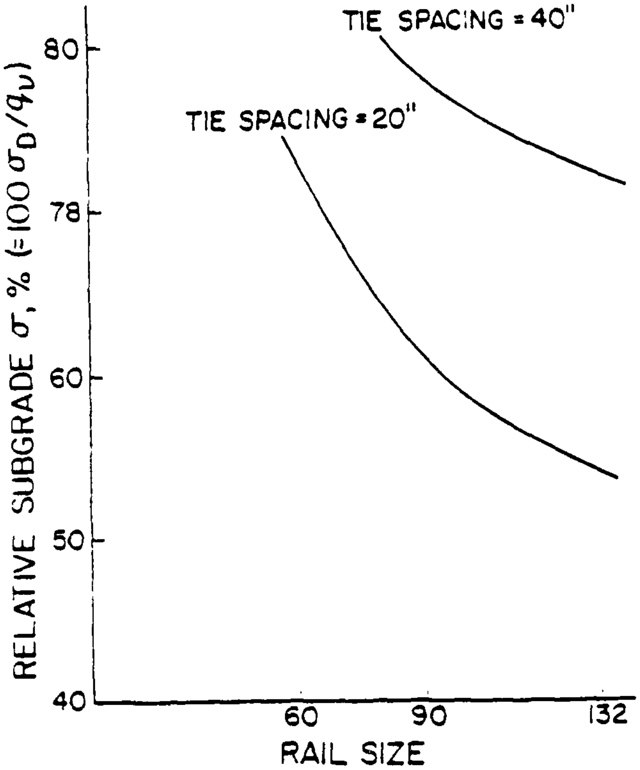



**Figure 3. Effect of tie spacing and rail size on relative subgrade stress.**

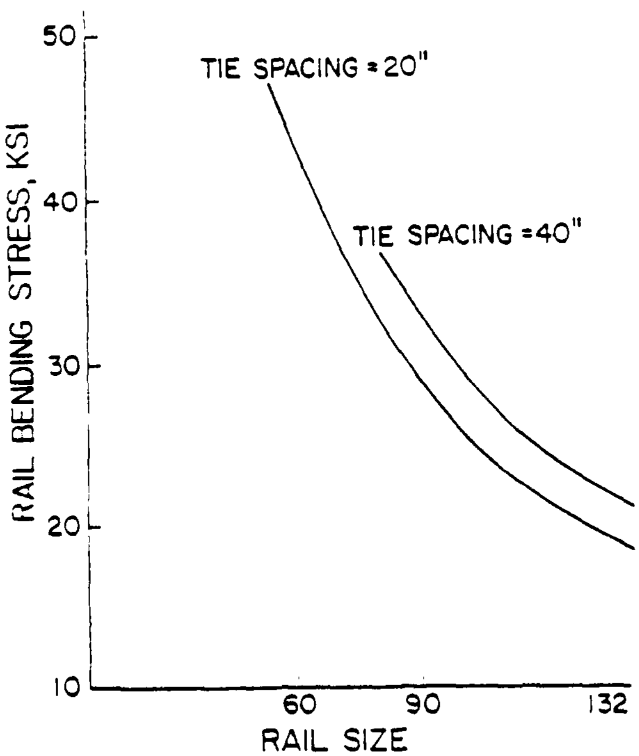



**Figure 4. Effect of tie spacing and rail size on rail bending stress.**

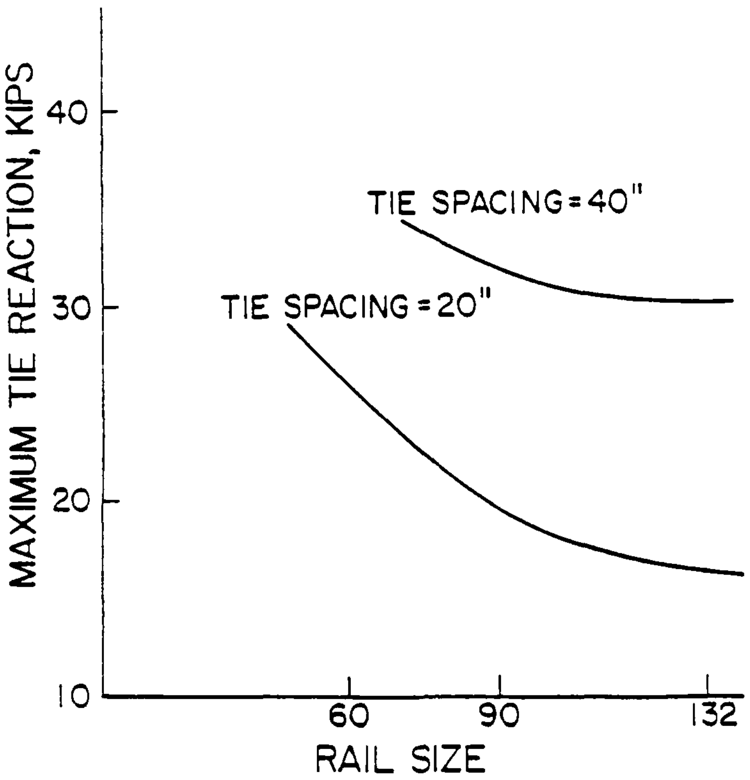


Figure 5. Effect of tie spacing and rail size on maximum ties reaction.

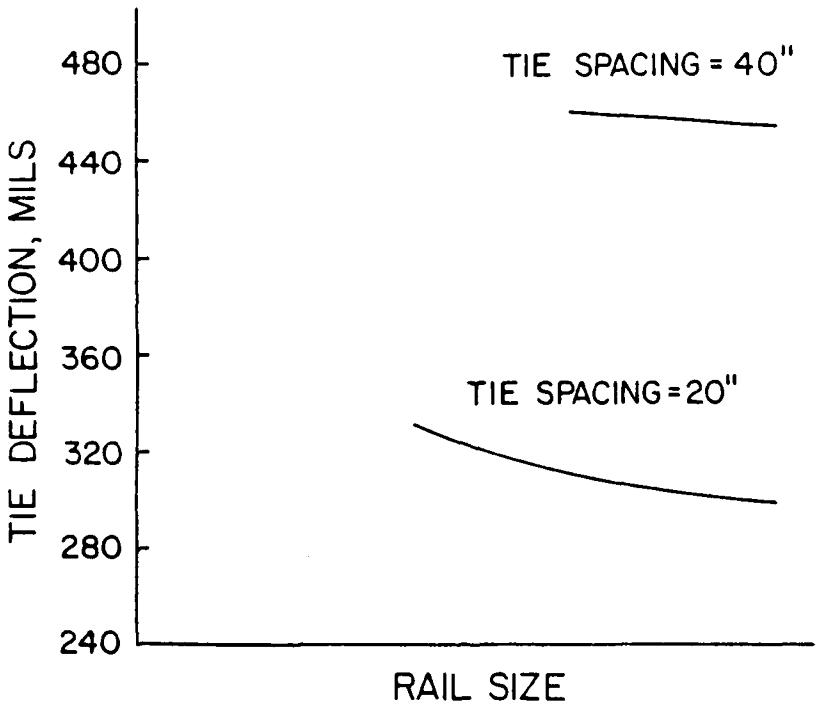


Figure 6. Effect of tie spacing and rail size on tie deflection.

## Operational Condition Evaluation

In many cases, gradual deterioration of the system components, localized defects, or improper track geometry may cause the operational failure of a track system. An effective maintenance management system can often correct these conditions before failure. These circumstances represent the track's operational condition.

A track segment's operational condition can be determined by measuring and inspecting the following:

1. Rail condition

2. Tie condition

3. Ballast/drainage condition

4. Subgrade/drainage condition

5. Track geometry.

### *Rail Condition*

Rail condition is determined by inspecting both internal and external defects. Internal defects, which must be detected with special equipment, are potentially hazardous because they cannot be seen and there are often no external indications of their presence. If undetected, an internal defect can grow until a rail break occurs. External defects include rail head wear (both top and side), corrosion, cracks, and various surface defects. These sometimes occur in combination with internal defects. In most cases, rail defects can be corrected by replacing the defective section.

### *Tie Condition*

Tie condition may be determined by combining a visual inspection procedure with calculations to produce a tie condition index. This index indicates the overall condition of ties in a given track segment.

Tie defects may cause the loss of both vertical and lateral rail support, leading to poor track geometry and loss of track load-carrying capacity. The need for tie replacement in a given track segment is determined by the number of defective ties, arrangement of defective ties (i.e., the presence of consecutive defective ties), and the severity of the defects. Figure 7 shows typical tie defects.

### *Ballast/Drainage Condition*

The ballast section holds the track in vertical and horizontal alignment. To perform this function properly, ballast must drain well and not suffer significant particle degradation. Visual inspection can be used to detect drainage problems and ballast deterioration. When such conditions exist, remedial action is required.

### *Subgrade/Drainage Condition*

Like the ballast section, the subgrade provides vertical track support. To do this, the subgrade must have sufficient strength and be properly drained. Visual inspection can be used to detect drainage problems and signs of subgrade failure.

*Track Geometry*

Track geometry is usually described by four parameters: gauge, crosslevel, alignment, and profile (Figure B1 of Appendix B). For military railroads or any other low-speed trackage, the most important geometric parameters are gauge and crosslevel. Ultimately, all track system components hold the rails in proper position, so a track geometry defect usually indicates the failure of one or more of these components.

Track geometry can be measured with simple devices on unloaded track. However, without full-scale loading, the results may not accurately reflect what the position of the rails would be under actual train traffic. This is especially true for track that is of light construction, is rarely used, has had minimal maintenance, or has structural defects. Since much of the Army's track falls into at least one of these categories, track geometry measurements should be taken with engine- or car-mounted devices.

Devices that mount on engine or car journals are available that allow measurements to be made and recorded continuously along the track under full-scale loads. This equipment is also easily installed and removed. A description of available devices is included in Appendix B.

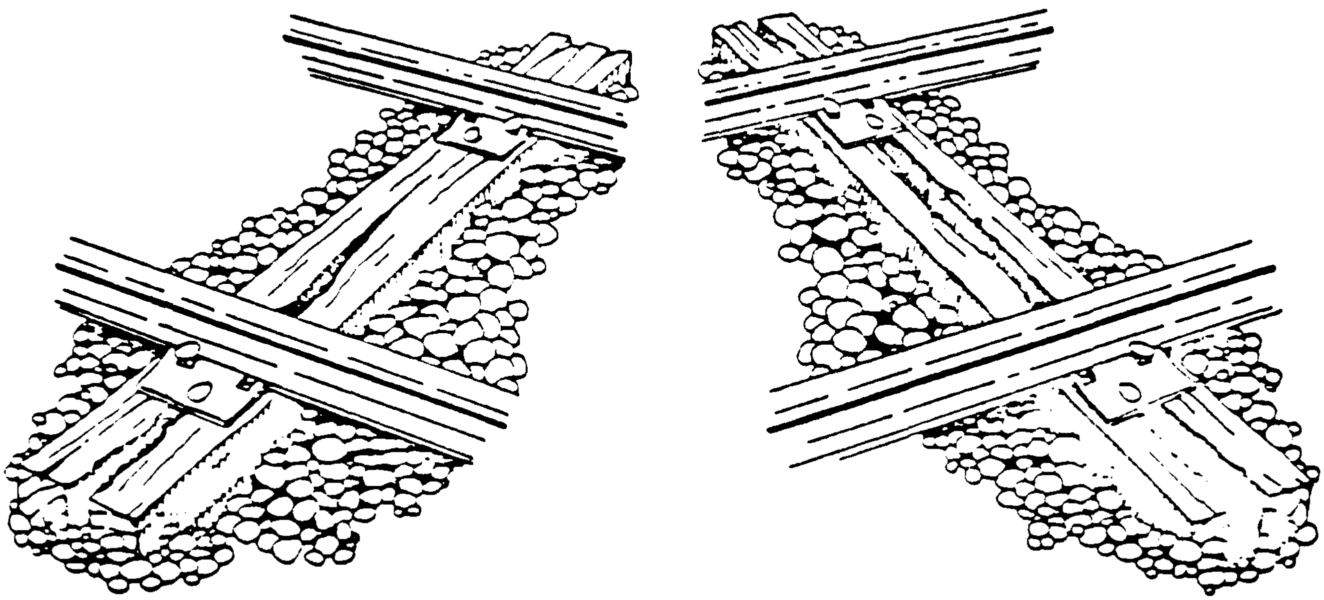
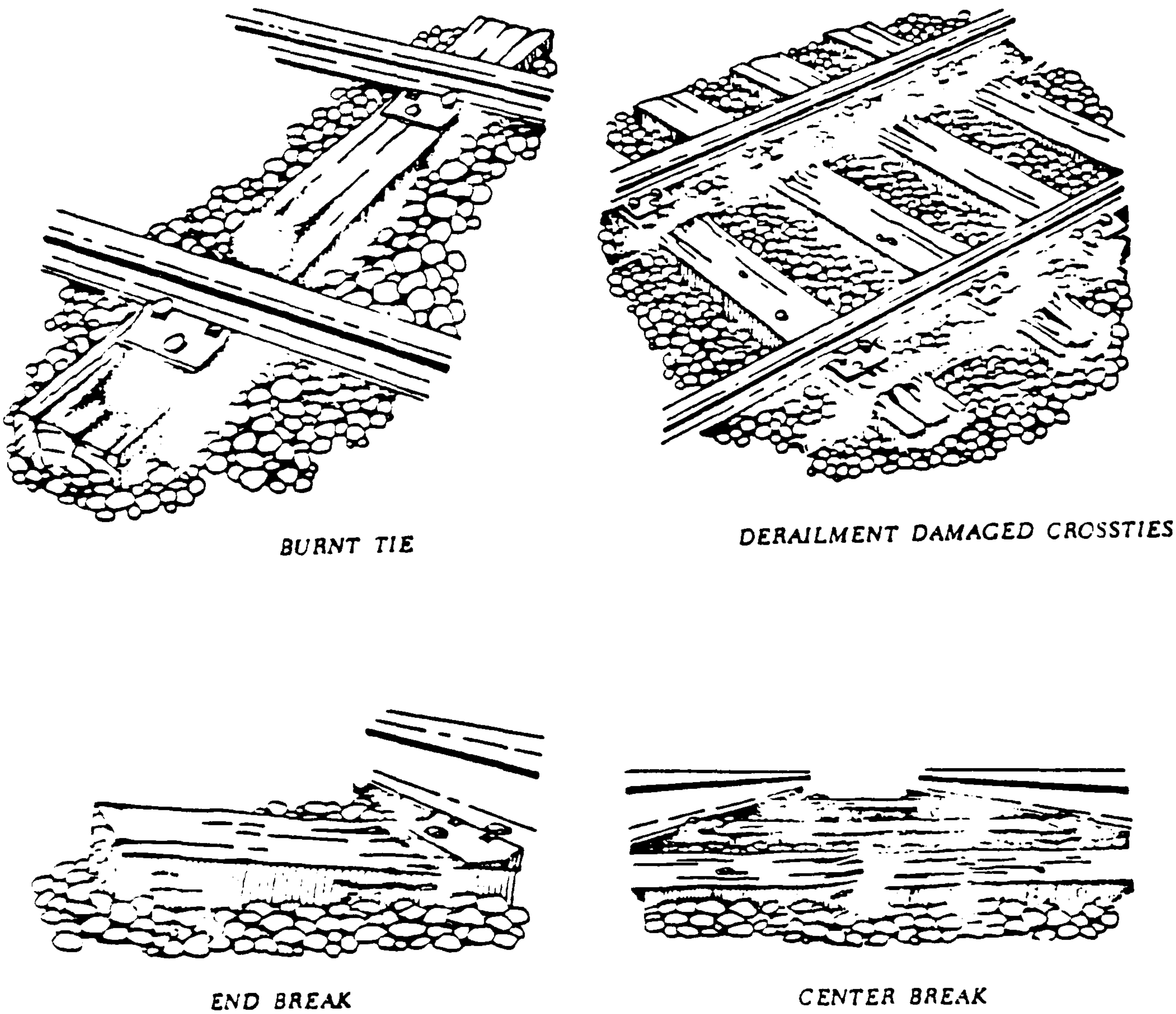



**Figure 7. Typical bad ties.**




BROKEN CROSSTIES

**Figure 7. (Cont'd)**

## 4 SUMMARY

This report has described the concepts developed for the proposed U.S. Army Railroad Track Maintenance Management System (RAILER). RAILER will consist of subsystems for network definition, data collection (including condition survey), data storage and retrieval, network data analysis, and project data analysis. Two major track evaluation categories have been identified: track structural condition and track operational condition. Evaluation procedures for use with RAILER have been recommended from these two categories.

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## METRIC CONVERSION FACTORS

| | |
|---|---|
| 1 in. | = 25.4 m |
| 1 mph | = 1.609 km/hr |
| 1 ft | = 0.3048 m |
| 1 mile | = 1.609 km |
| 1 psi | = 703.070 $kg/m^2$ |
| 1 kip | = 453.5 kg |
| 1 mil | = 0.0254 mm |
| 1 lb | = 0.4535 kg |
| 1 ton | = 907.185 kg |
| 1 ksi | = $703.07 \times 10^3$ $kg/m^2$ |

APPENDIX A:

# DEVELOPMENT OF FOUNDATION CONDITION/TRACK STABILITY CONCEPTS*

## 1 INTRODUCTION

The typical rail-tie system is supported by a ballast layer, and in some cases, by subballast which overlies the subgrade. Each component must have certain characteristics if the overall system is to provide adequate track structure performance.

The Track Foundation Structural Condition Index (FCI) is designed to quantify the condition of the ballast, subballast, and subgrade. "Deduct values" are used to relate the negative effect of various factors. The initial intent of the system was to base FCI inputs on visual inspection and/or on simple mechanical measurements (only hand-held-type devices).

If a track support system (ballast-subballast-subgrade) is providing functionally satisfactory performance, the track settlement--million gross tons traffic plot is approximately linear, indicating "stable" behavior. If a component of the track support system is "overstressed," track settlement may increase rapidly (unstable behavior), prompting the need for maintenance. Figure A1 shows "stable" and "unstable" track settlement responses.

Shear strength and repeated loading behavior (resilient modulus, permanent deformation resistance) are the engineering properties of the granular materials (ballast) and subgrade soils that control track support system structural response and performance. Gradation, plasticity (liquid limit (LL), plastic limit (PL), PI, moisture content, and density are the main characteristics that influence shear strength and repeated loading behavior. Particle shape, angularity, and surface texture are other important granular material characteristics.

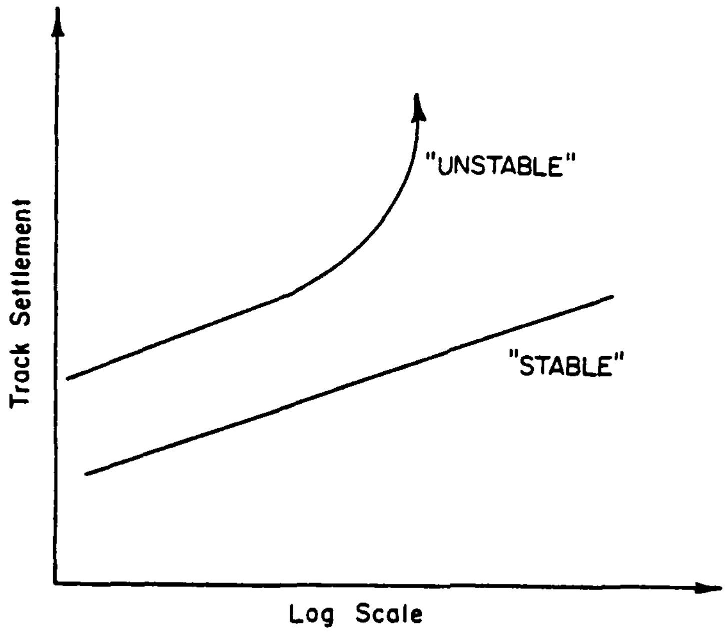
Moisture content and availability influence the shear strength and repeated loading behavior of soils and granular materials. Moisture content is not a "static" value, but rather varies with time. The track section's moisture regime must be well-defined: water table depth, surface drainage (including ditches), permeability/drainage properties, and soil-moisture characteristics (water content-suction) are important factors that influence the moisture regime.

Factors such as gradation, plasticity properties, thickness, aggregate geometric properties (shape, angularity, surface texture) do not change significantly as a function of time. In contrast, moisture content/availability vary greatly, depending on the moisture regime.

## 2 TRACK DEFLECTION/TRACK PERFORMANCE RELATIONS

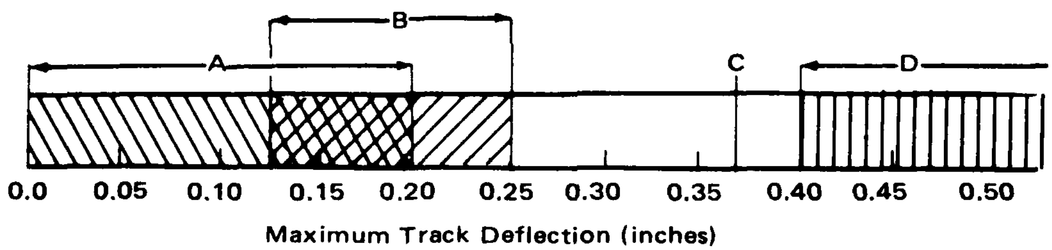
Track deflection is frequently used to "quantify" track strength. Figure A2 shows a typical "track deflection/track performance" relation. High deflections are associated with increased "relative subgrade stress" levels (subgrade stress/subgrade strength). High subgrade stress levels produce increased rates of subgrade permanent strain accumulation. Thus, track geometry deteriorates more rapidly.

*Prepared by Marshall R. Thompson, P.E., Urbana, IL.




• MGT of Traffic
• Number of Load Repetitions

**Figure A1.** **"Stable" and "unstable" track settlement performance.**




| Range | Track Behavior |
|---|---|
| A | Deflection range for track which will last indefinitely. |
| B | Normal maximum desirable deflection for heavy track to give requisite combination of flexibility and stiffness. |
| C | Limit of desirable deflection for track of light construction ($\leqslant$100 lb). |
| D | Weak or poorly maintained track which will deteriorate quickly. |

Values of deflection are exclusive of any loosenness or play between rail and plate or plate and tie and represent deflections under load.

**Figure A2.** **Track deflection/track performance relations.** (From J. R. Lundgren, G. C. Martin, and W. W. Hay, *A Simulation Model of Ballast Support and the Modulus of Track Elasticity*, Civil Engineering Studies, Transportation Engineering Series No. 4 [University of Illinois, September 1970].)

The factors that control track deflection (for given loading, rail size, and tie spacing) are the thicknesses and quality of the granular ballast and subballast layers and subgrade modulus and strength.

*Track sections* with differing ballast, subballast, and subgrades may display the same track deflection under load, but (depending on the shear strength and permanent deformation characteristics of the granular materials and soils) they can display different track system performance. Nevertheless, track deflection is still the most reliable single indicator of track strength and potential track system performance.

## 3 FCI INPUT DEVELOPMENT

Initial directives for the FCI development indicated that inputs should be based on "visual" and/or simple mechanical measurements. Thus, "actual" test property data (PI, LL, strength, gradation, etc.) or "estimated" properties (based on visual observations, etc.) can be used.

An adequate assessment of the ballast-subballast-subgrade system must provide "quantitative" measures of the shear strength and the repeated loading (resilient moduli, permanent deformation) behavior of the granular materials and subgrade soil.

For granular materials, the important influencing factors are:

Gradation
Geometry of fines (LL, PL, PI)
Layer thickness(es)
Density
Moisture content.

For fine-grained cohesive soils, the important factors are:

Gradation
Plasticity (LL, PL, PI)
Moisture content
Density.

Resilient moduli and repeated loading behavior characteristics may be sensitive to minor variations in gradation, plasticity, density, and moisture content. Thus, it is very hard to accurately quantify the important factors listed previously.

Resilient moduli and permanent deformation behavior (repeated loading properties) are also "stress-dependent." For granular materials, resilient moduli increase as stress state increases; however, for fine-grained cohesive soils, resilient moduli decrease with increasing stress. For both granular and fine-grained soils, permanent deformation accumulation rates increase for higher repeated stress levels. Thus, track loading/rail/tie factors influence track deflection and track geometry deterioration (permanent deformation in ballast/subballast/subgrade) for a given track foundation condition. FCI acceptance levels should therefore be related to track loading/rail/tie conditions.

Standardized tests are available for determining certain FCI inputs. Table A1 lists the appropriate *American Society for Testing and Materials (ASTM)* standards. Other "simple" field tests can be conducted to characterize in-situ strength, particularly for fine-grained cohesive soils.

Table A1

Standardized Tests for FCI Inputs

| Property | ASTM Standard | |
|---|---|---|
| Gradation | D422: | Particle-Size Analysis of Soils |
| Plasticity | D423: | Liquid Limit of Soils |
| | D424: | Plastic Limit and Plasticity Index of Soils |
| Moisture Content | D2216: | Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures |

Procedures have been developed for describing soils based on visual examination and simple manual tests. Chapter 2 of Army Technical Manual 5-530, *Materials Testing*, describes procedures for "field classification" leading to a unified classification. ASTM D2488, *Description of Soils (Visual-Manual Procedure)*, is also available. Many "general property relations" between aggregate/soil materials and soil classification have been developed.

The accuracy/precision of an FCI value for a given track segment obviously depends on the available inputs. Direct measurement procedures are more reliable and definitive.

## 4 BALLAST AND SUBBALLAST

Shear strength and repeated loading behavior (resilient moduli, permanent deformation resistance) are the major properties of interest when rating ballast and subballast. The two are similar in that factors that increase granular material shear strength also increase permanent deformation resistance.

Gradation and aggregate geometric properties (shape, angularity, surface texture) can be used to estimate shear strength and repeated loading behavior of granular materials. If excessive fines are present, PI should be considered. For a given aggregate material, density greatly influences shear strength and permanent deformation resistance.

Resilient moduli values are less sensitive to density variation and gradation factors. In general:

1. Larger maximum size increases resilient moduli.

2. More densely graded materials display less resilient moduli.

3. For a given gradation, generic rock type has a minor effect on resilient moduli.

4. Gravelly-type materials (rounded, less angular, more polished surface texture) are more "stress-level sensitive" than crushed stone materials.

Visual evaluation (gradation, shape, angularity, surface texture) and simple manual procedures (PI of fines) can be used to broadly group granular materials. Unfortunately, in-situ density cannot be evaluated conveniently or easily.

## 5 SUBGRADE STRENGTH

Subgrade strength and moduli are important FCI inputs. In-situ subgrade soil strength is *quite variable*. Soil type and moisture content are the primary factors influencing strength. Most problems related to subgrade soil strength are associated with moisture contents that are "wet of optimum." Granular-type subgrades (gravel, sand, etc.) are seldom a source of concern. The major emphasis in FCI is fine-grained cohesive soils.

Relevant strength/moduli data can be (1) directly measured by a hand-held device, (2) estimated from general strength-soil classification relations, and (3) estimated from data such as textural composition (sand, silt, clay fractions), plasticity, and in-situ water content.

### Hand-Held Strength-Measuring Devices

One person can easily use a suitable hand-held device to perform FCI measurements. The standard Corps of Engineers cone penetrometer, drive cone penetrometer, pocket penetrometer, and vane-shear devices (including TOR-VANE type equipment) could be used. The major obstacle to evaluating the in-situ soil in a track structure is gaining easy access to the subgrade layer beneath the rail seat area. An alternate (and much less desirable approach) is to take the measurements in more readily accessible areas such as the shoulder. Only the heavier "drive cone penetrometer" type devices might be driven through granular layers into the subgrade zone. A "heavy" drive cone penetrometer has been used successfully by South Africa in pavement evaluation activities.[5]

In most cases, general correlations have been developed that relate cone index, California Bearing Ratio (CBR), shear strength, moduli, etc.; this facilitates the development of input data in "common terms" (for example, soil shear strength). Table A2 shows several correlations.[6]

### "Manual" Strength Tests

Manual tests that do not require any equipment can be used to estimate shear strength. Table A3 shows ASTM D2488 relations; Table A4 shows similar, but more detailed relations proposed by Peck, et al.[7]

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[5] E. G. Kleyn and P. F. Savage, "The Application of the Pavement DCP to Determine the Bearing Properties and Performance of Road Pavements," *Proceedings, Vol 1, Bearing Capacity of Roads and Airfields* (The Norwegian Institute of Technology, Trondheim, Norway, 1982).

[6] M. L. Traylor and M. R. Thompson, *Sinkage-Prediction-Subgrade Stability*, Civil Engineering Studies, Transportation Engineering Series No. 17 (University of Illinois, June 1977).

[7] R. B. Peck, W. E. Hanson, and T. H. Thornburn, *Foundation Engineering* (John Wiley & Sons, Inc., 1974).

## Table A2

**Empirical Strength Correlations***

1. $S = \frac{CI}{12.5}$
2. $S = 7 + 0.02\ CI$
3. $S = 2.25\ CBR$
4. $CI = 50\ CBR$
5. $CI = 32\ CBR$
6. $CI = 8\ q_u$
7. $CBR = \frac{CI}{50} - 0.8$
8. $CBR = \frac{CI}{33} - 0.03$
9. $CBR = \frac{q_u}{4.5}$
10. $CBR = 0.15\ q_u - 0.2$
11. $CBR = 0.18\ q_u - 1.5$

*Notes: S = shear strength, psi (unconfined compressive strength/2)
CI = Cone Index (U.S. Army Corps of Engineers)
CBR = California Bearing Ratio
$q_u$ = unconfined compressive strength, psi

## Table A3

### Identifying Consistency of Fine-Grained Soils From Manual Tests (ASTM D2444)

| Consistency | Identification procedure | Shear strength, psi | Unconfined compressive strength, psi |
|---|---|---|---|
| Soft | Easily penetrated several inches by thumb | <3.5 | <7.0 |
| Firm (medium) | Penetrated several inches by thumb with moderate effort | 3.5-7.0 | 7.0-14.0 |
| Stiff | Readily indented by thumb, but penetrated only with great effort | 7.0-14.0 | 14.0-28.0 |
| Very stiff | Readily indented by thumbnail | 14.0-28.0 | 28.0-56.0 |
| Hard | Indented with difficulty by thumbnail | 28.0-56.0 | >56 |

## Table A4

### Qualitative and Quantitative Expressions for Consistency of Clays

(From R. B. Peck, W. E. Hanson, and T. H. Thornburn, *Foundation Engineering* [John Wiley and Sons, Inc., 1974])

| Consistency | Identification procedure | Unconfined compressive stength $q_u$ (psi) |
|---|---|---|
| Very soft | Easily penetrated several inches by fist | <3.5 |
| Soft | Easily penetrated several inches by thumb | 3.5 - 7.0 |
| Medium | Can be penetrated several inches by thumb with moderate effort | 7.0 - 14.0 |
| Stiff | Readily indented by thumb but penetrated only with great effort | 14.0 - 28.0 |
| Very stiff | Readily indented by thumbnail | 28.0 - 56.0 |
| Hard | Indented with difficulty by thumbnail | >56 |

## Estimating Soil Strength

Reasonable estimates of subgrade soil strength can be based on soil classification and "moisture state" data. Freeze-thaw action greatly reduces a soil's strength/modulus.

Transportation Road Research Laboratory (TRRL): Table A5 and Figure A3 show British procedures[8] for estimating in-situ soil strength for "equilibrium" moisture conditions. Input factors are soil texture and water table depth. Note that for high water table conditions, the estimated CBR for fine-grained soils is 2 or less.

WES Cone Index: Based on extensive mobility studies at the U.S. Army Engineer Waterways Experiment Station (WES)[9], general relations have been proposed for estimating in-situ strength (CI values). Unified or U.S. Department of Agriculture (USDA) soil classification, topography, and water table depth information are required inputs.

Emphasis is placed on moisture conditions that are "wet of optimum." If a fine-grained cohesive soil has a CI value less than 300, moisture content is generally greater than optimum.[10] For wet-of-optimum conditions, a given soil's strength is controlled mainly by moisture content.

General topography/moisture regime conditions considered in the WES procedure are:

1. High topography, wet season
2. Low topography, wet season
3. Low topography, high moisture.

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[8]"A Guide to the Structural Design of Pavements for New Roads," *Road Note 29*, third ed. (Road Research Laboratory, England, 1970).

[9]R. L. Wright and J. R. Burn, *Mobility Environmental Research Study - A Quantitive Method for Describing Terrain for Ground Mobility, Volume 11, Surface Composition*, Technical Report No. 3-726 (U.S. Army Engineer Waterways Experiment Station [WES], January 1968); *Trafficability of Soils - Soil Classification*, Technical Memorandum No. 3-240, Sixteenth Supplement (WES, August 1961); S. J. Knight and A. A. Rula, *Measurement and Estimation of the Trafficability of Fine-Grained Soils*, Miscellaneous Paper No. 4-441 (WES, November 1961); S. J. Knight, *Some Factors Affecting Moisture Content-Density-Cone Index Relations*, Miscellaneous Paper No. 4-457 (WES, November 1961); *A Limited Study of Factors that Affect Soil Strength*, Miscellaneous Paper No. 4-284 (WES, August 1958); *A Summary of Trafficability Studies through 1955*, Technical Memorandum No. 3-240, Fourteenth Supplement (WES, December 1956); W. C. Grenke, *Observing, Analyzing and Forecasting the State of the Ground*, Contract Report 3-112, prepared by Wilson, Nuttal, Raimond Engineering Inc. (WES, May 1965); *Soil Trafficability*, Technical Manual 5-330/Air Force Manual 86-3, Vol 11 (Departments of the Army and Air Force); J. G. Collins, *Forecasting Trafficability of Soils; Report 10, Relations of Strength to Other Properties of Fine-Grained Soils and Sands with Fines*, Technical Memorandum No. 3-331 (WES, July 1971).

[10]S. J. Knight.

## Table A5

**British Classification of Subgrades for Concrete Pavement Design**
(From "A Guide to the Structural Design of Pavements for New Roads," *Road Note 29*, third ed. [Road Research Laboratory, England, 1970].)

| Subgrade Class | Definition |
|---|---|
| Weak | All subgrades of CBR value 2 percent or less as defined below |
| Normal | Subgrades other than those defined by other categories |
| Very stable | All subgrades of CBR value 15 percent or more as defined below. This category includes undisturbed foundation of old roads. |

**Estimated Laboratory CBR Values for British Soils Compacted at the Natural Moisture Content**

| | | Depth of water table below subgrade surface | |
|---|---|---|---|
| Type of soil | Plasticity index | More than 24 in. | 24 in. or less |
| Heavy clay | 70 | 2 | 1 |
| | 60 | 2 | 1.5 |
| | 50 | 2.5 | 2 |
| | 40 | 3 | 2 |
| Silty clay | 30 | 5 | 3 |
| Sandy clay | 20 | 6 | 4 |
| | 10 | 7 | 5 |
| Silt | -- | 2 | 1 |
| Sand (poorly graded) | Nonplastic | 20 | 10 |
| Sand (well-graded) | Nonplastic | 40 | 15 |
| Well-graded sandy gravel | Nonplastic | 60 | 20 |

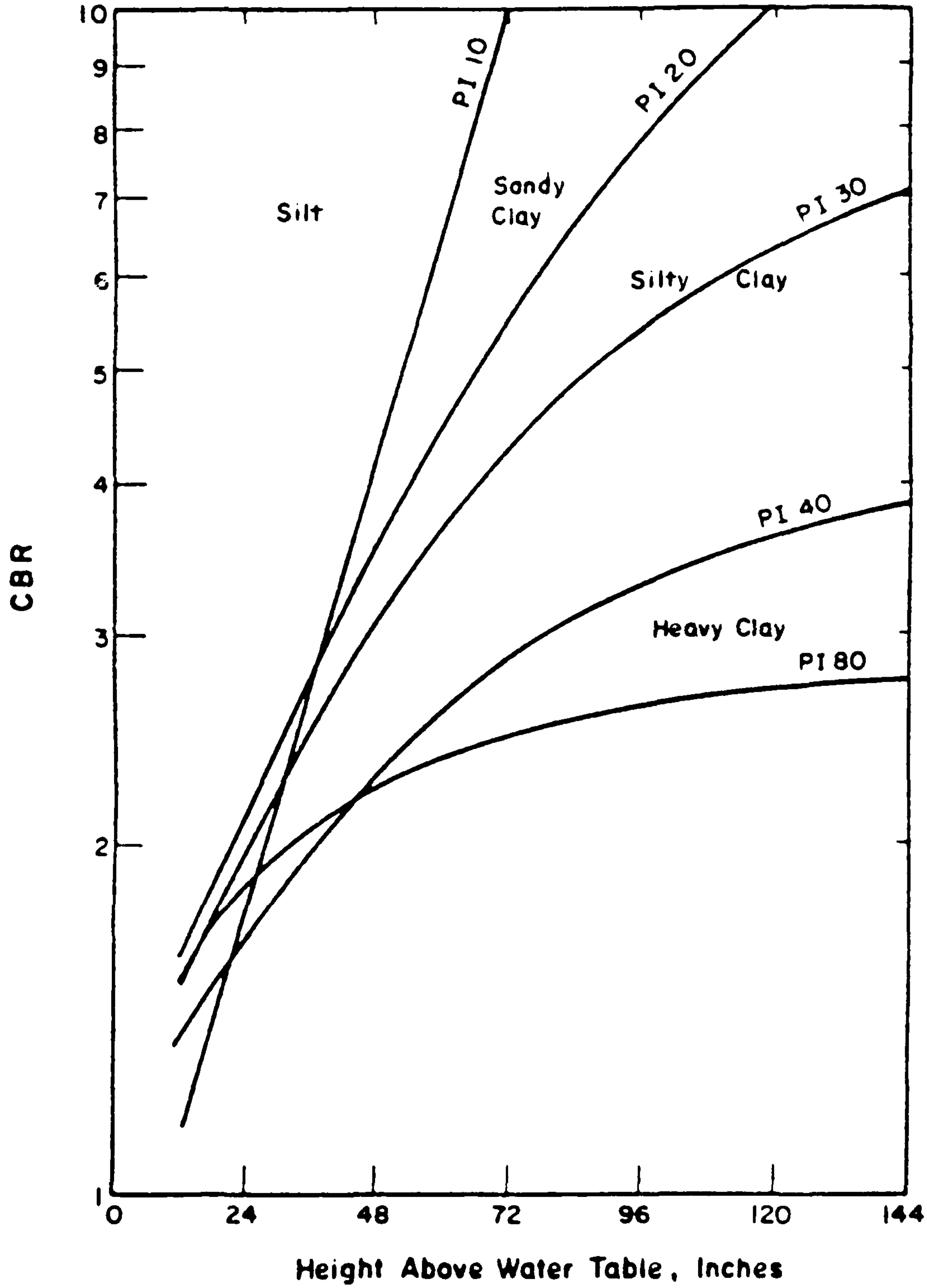


Figure A3. CBR water table relations for fine-grained soils.

Figure A4 illustrates the various conditions. The definitions below may also help visualize these conditions.

High topography: Sites of high topography have water tables at depths greater than 4 ft from the surface at all times.

Low topography: A site of low topography is one at which a water table is known to exist within 4 ft of the surface, perenially or at some time during the year.

Wet season conditions: The wet season condition is intended to represent the average condition prevailing in soils during the wet season.

High moisture condition: The high moisture condition represents the worst trafficability condition that can occur at sites undergoing seasonal changes.

In Figure A4, the dry season for temperate and humid climates is the period from May to November, and the wet season is from November to May.[11] Figure A5 gives more specific "wet season" information.

The most extensive strength/soil type data available were gathered in a WES study[12] which considered 310 separate sites throughout the United States. Figures A6 and A7, developed from the data obtained, show the relations between soil type, cone index, and rating cone index. Table A6 gives tabulations (decreasing order) of mean cone index and mean remolding index for the various soil types for wet season conditions.

Strength-Classification Relations: Soil can be classified based on visual or other simple manual procedures. Table A7 and Figure A8 provide various general soil strength/soil classification relations.

**Soil Survey Information**

County Soil Survey reports are available for many areas of the United States. Additional information, such as soil association maps, are also often available. Soil series identification is very useful information. Besides textural classification data, these sources also provide internal soil drainage factors. Thus, the two major factors (soil texture and moisture regime) that influence strength can be considered based on soil series. Some engineering test data are also generally provided in recent County Soil Reports.

If this type of report or any other source of soil series information is available, they should be used, since the data they provide are of great value.

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[11] *A Summary of Trafficability Studies through 1955.*
[12] *Trafficability of Soils - Soil Classification.*

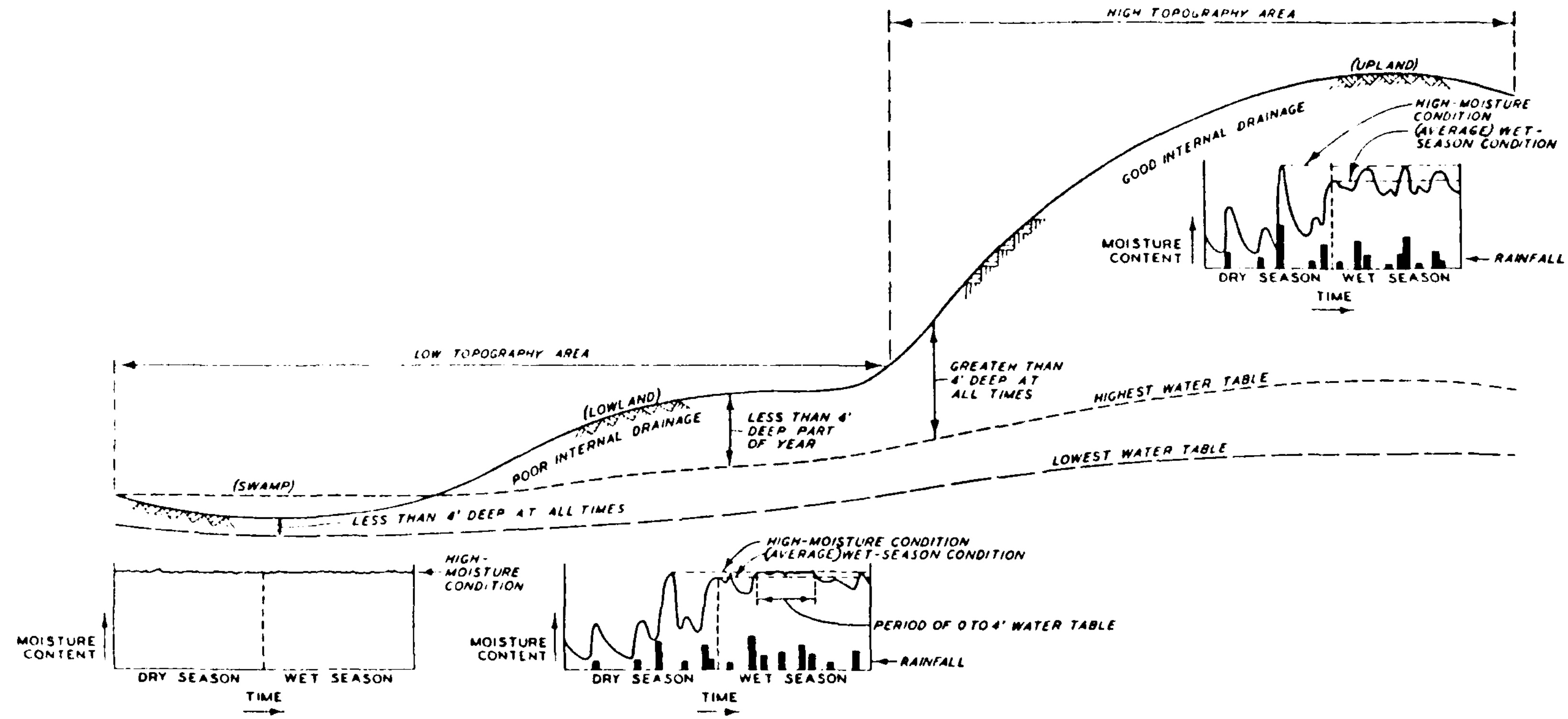


**Figure A4. Profile of a typical area showing various topographic moisture conditions during the year.** (From *Trafficability of Soils - Soil Classification*, Technical Memorandum No. 3-240, Sixteenth Supplement [WES, August 1961].)

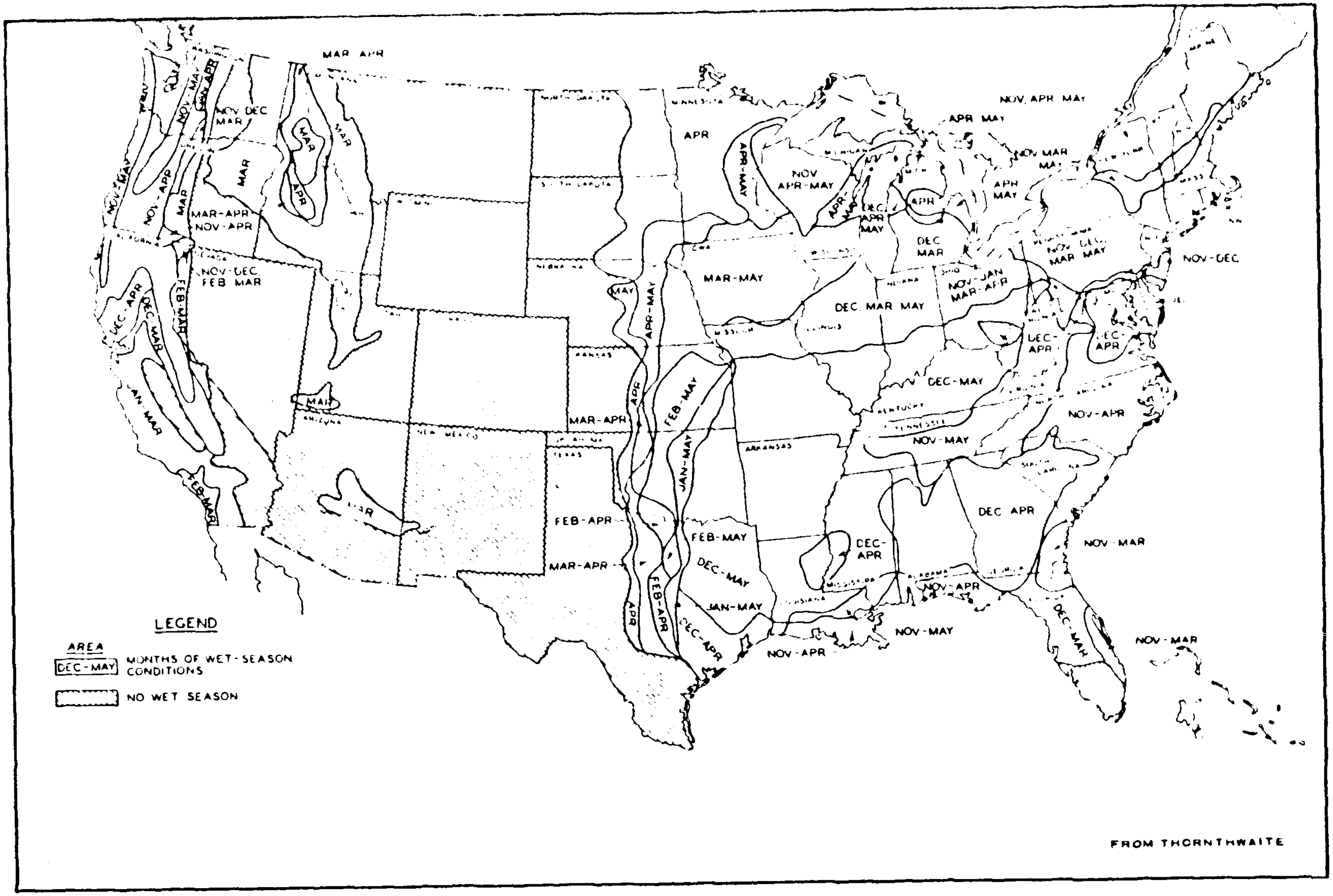


Figure A5. **Distribution and months duration of wet-season conditions.** (From *Trafficability of Soils - Soil Classification*, Technical Memorandum No. 3-240, Sixteenth Supplement [WES, August 1961].)

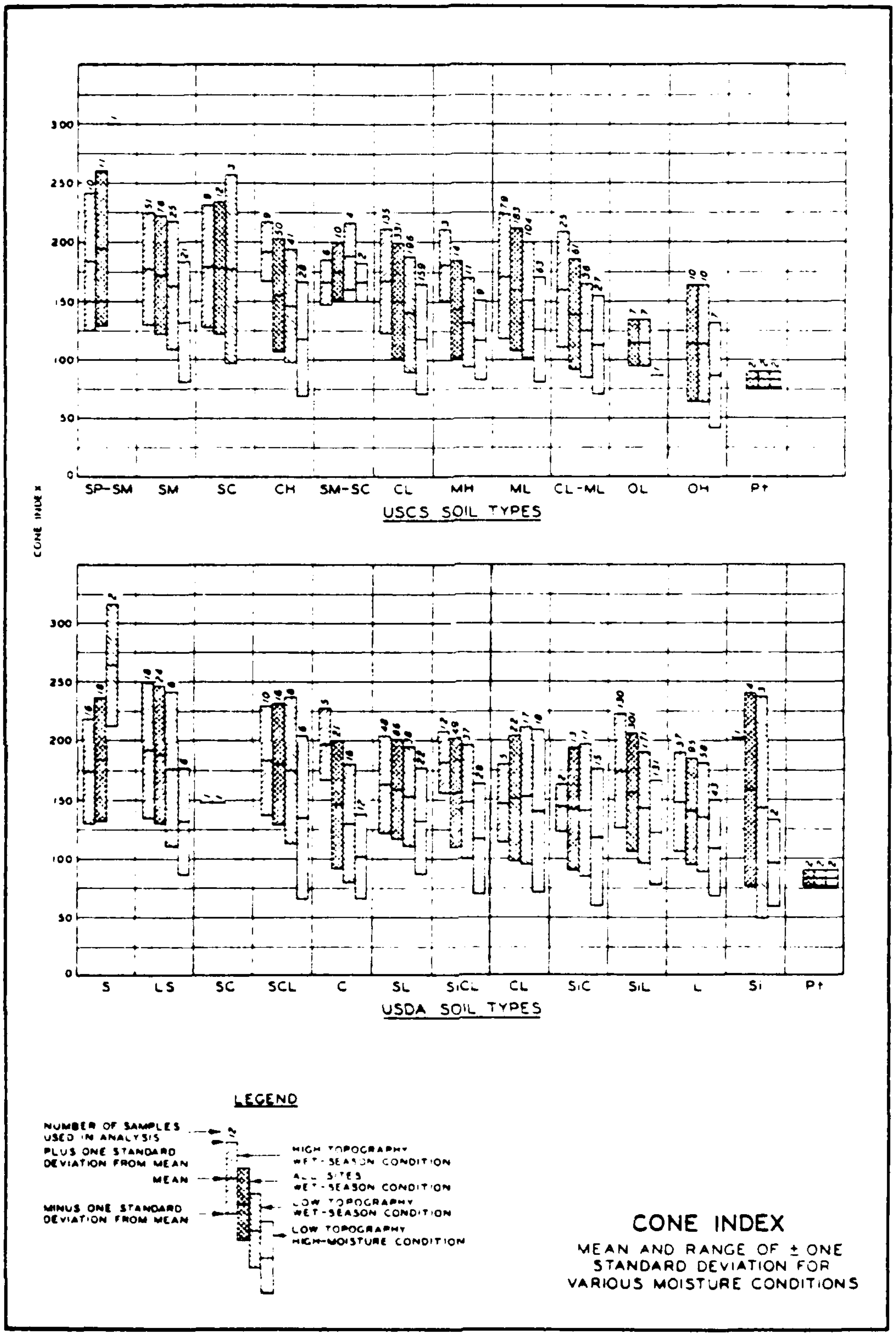


**Figure A6. Relation between soil type and cone index.** (From *Trafficability of Soils - Soil Classification*, Technical Memorandum No. 3-240, Sixteenth Supplement [WES, August 1961].)

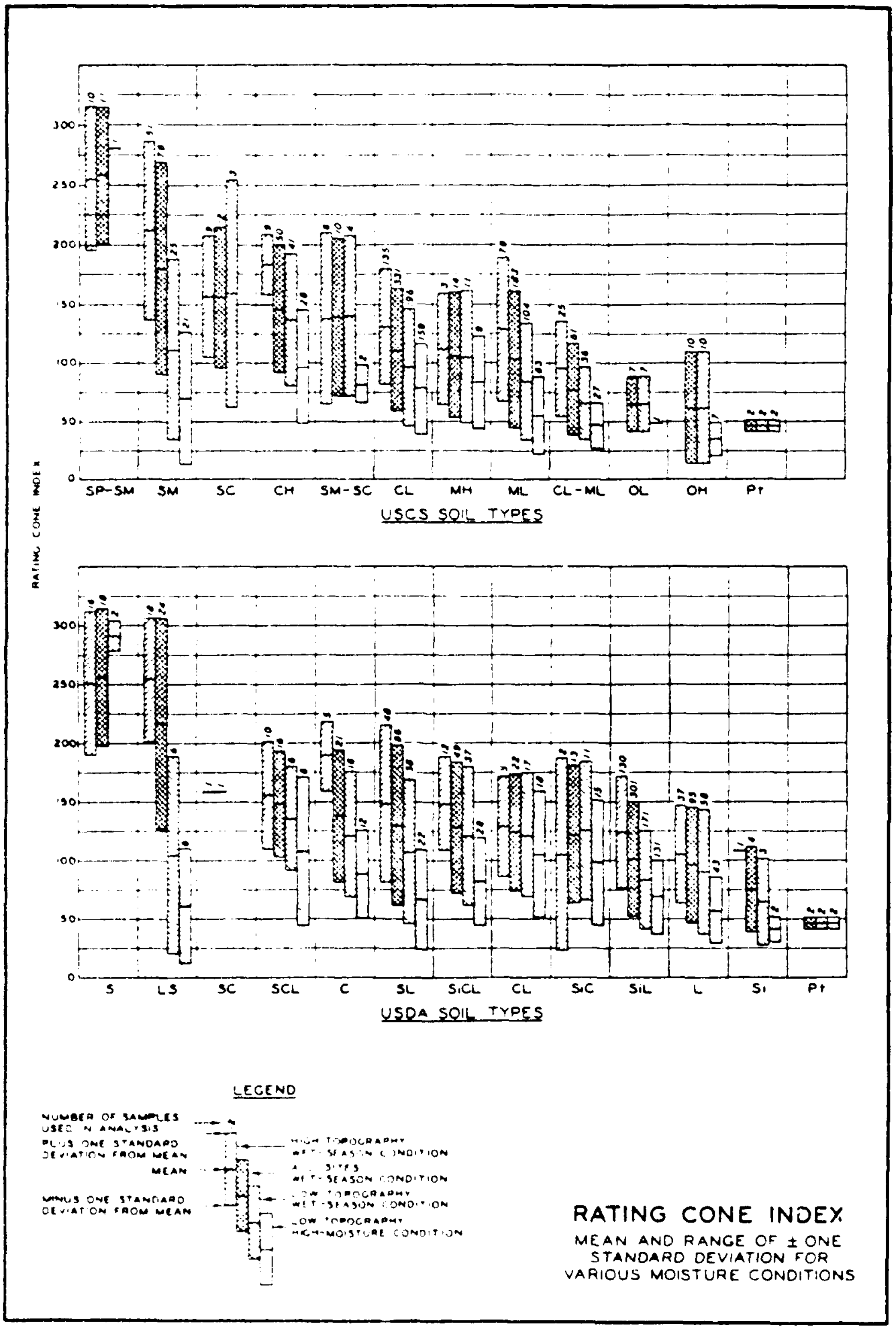


Figure A7. **Relation between soil type and rating cone index.** (From *Trafficability of Soils - Soil Classification*, Technical Memorandum No. 3-240, Sixteenth Supplement [WES, August 1961].)

Table A6

**Mean Values for Cone Index and Remolding Index (Wet Season Condition)**

**Cone index:**

| USCS type | Mean cone index | USDA type | Mean cone index |
|---|---|---|---|
| SP-SM | 194 | LS | 188 |
| SC | 178 | S | 184 |
| SM-SC | 175 | SCL | 180 |
| SM | 172 | SL | 159 |
| ML | 160 | Si | 158 |
| CH | 155 | SiL | 156 |
| CL | 150 | SiCL | 156 |
| MH | 143 | CL | 151 |
| CL-ML | 139 | SC | 148 |
| OL | 115 | C | 146 |
| OH | 114 | SiC | 142 |
| Pt | 83 | L | 140 |
| | | Pt | 83 |

**Remolding Index:**

| USCS type | Mean remolding index | USDA type | Mean remolding index |
|---|---|---|---|
| SP-SM | 1.61 | S | 1.61 |
| SM | 1.09 | LS | 1.24 |
| CH | 0.95 | SC | 1.07 |
| SC | 0.86 | C | 0.96 |
| SM-SC | 0.84 | SiC | 0.85 |
| MH | 0.73 | SL | 0.84 |
| CL | 0.71 | SCL | 0.83 |
| ML | 0.63 | CL | 0.81 |
| OL | 0.56 | SiCL | 0.79 |
| Pt | 0.56 | L | 0.66 |
| OH | 0.55 | SiL | 0.63 |
| CL-ML | 0.54 | Pt | 0.56 |
| | | Si | 0.47 |

## Table A7

**Soil Strength/Soil Classification Relations**
**Illinois Department of Transportation CBR - American Association of State Highway and Transportation Officials (AASHTO) Classification and CBR-K Relations**
(From *Section 7-Pavement Design* [Illinois Department of Transportation, May 1982].)

| AASHTO class | CBR |
|---|---|
| A-1 | 20 |
| A-2-4; A-2-5 | 15 |
| A-2-6; A-2-7 | 12 |
| A-3 | 10 |
| A-4; A-5; A-6 | 3 |
| A-7-5; A-7-6 | 2 |

| CBR | K |
|---|---|
| 50 | 500 |
| 20 | 250 |
| 10 | 200 |
| 5 | 140 |
| 3 | 100 |
| 2 | 70 |

**Unified Soil Classification--Modulus of Subgrade Reaction Relations** (From *Materials Testing*, Army Technical Manual 5-530, Navy NAVFAC MO-330, and Air Force Manual No. 89-3 [Departments of the Army, Navy, and Air Force, 1971].)

| Unified soil classification | Modulus of subgrade reaction, psi/in. |
|---|---|
| GW; GP; GM (LL<28, PI<6) | 300 or more |
| GM; (LL>28); GC; SW; SP; SM; SC | 200-300 |
| ML; CL; OL; MH | 100-200 |
| CH; OH | 50-100 |

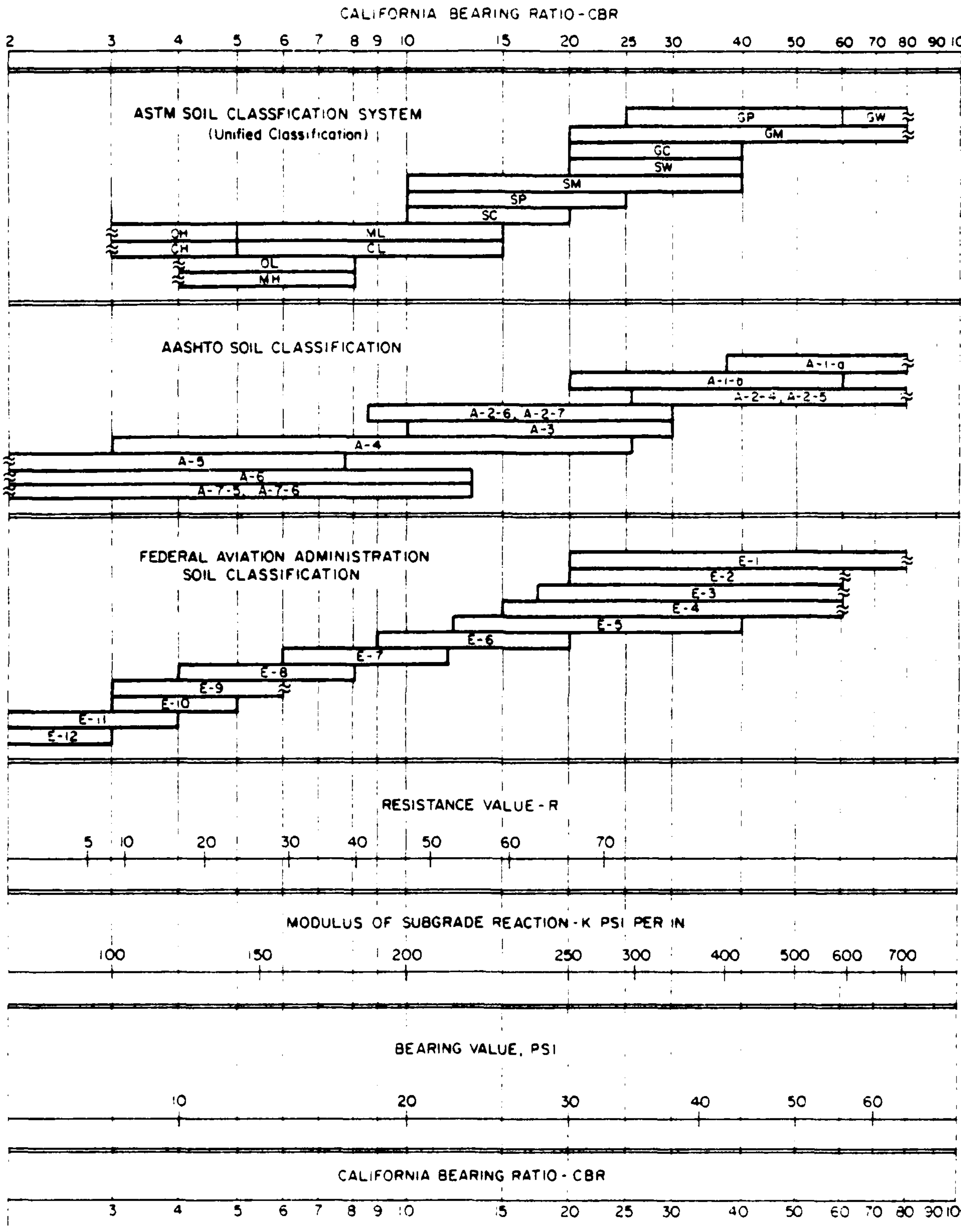


**Figure A8. Soil classification/soil strength correlations (as summarized by the Portland Cement Association).** (From *Soil Primer* [Portland Cement Association, 1973].)

## Summary

Several techniques can be used to evaluate subgrade strength for FCI purposes. Direct measurement is more precise than estimating. If estimation is used, preference should be given to a procedure that considers both texture and moisture content.

## 6 FCI RATING SYSTEM

The proposed FCI is based on the proposition that track deflection is a valid indicator of track structure, strength, and performance. For given loading/rail/tie conditions, layer thickness and granular material characteristics of the ballast and subballast and subgrade strength/modulus properties are the factors that control track deflection. The FCI rating is only for current conditions. If granular material and subgrade soil properties change, track deflection will be influenced, and FCI will change.

A series of ILLI-TRACK analyses was conducted to develop typical track response data for a given set of conditions (rail size, tie size and spacing, and axle loading). Ballast depths of 6, 12, 18, and 24 in. and subgrade conditions of medium, soft, and very soft were considered. Table A8 summarizes pertinent track and loading data and ballast and subgrade soil properties. Table A9 summarizes track and subgrade response data.

These publications provide documentation and references for the ILLI-TRACK computer program.[13] ILLI-TRACK has been used for various studies sponsored by the Association of American Railroads, materials suppliers, and several railroads. ILLI-TRACK is unique in that it can consider the entire track support system (ballast/subballast/subgrade). This contrasts to conventional track analysis procedures which use a "track modulus" value to characterize the entire track support system. ILLI-TRACK realistically models linear elastic materials (such as cement-treated subballast material) and stress-dependent materials (granular, fine-grained soils). Mohr-Coulomb failure criteria (C, $\Phi$) are used for granular materials and fine-grained soils.

The ILLI-TRACK response data (Table A9) were used to develop the track reflection, thickness, strength stress plot presented in Figure A9. Relative subgrade stress is the maximum subgrade deviator stress divided by the unconfined compressive strength (twice the cohesion if $\Phi = 0$).

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[13] S. D. Tayabji and M. R. Thompson, *Program ILLI-TRACK A Finite Element Analysis of Conventional Railway Support System - User's Manual and Program Listing*; S. D. Tayabji and M. Thompson, "Considerations in the Analysis of Conventional Railway Track Support System," *Transportation Engineering Journal*, Vol 103, No. TE2 American Society of Civil Engineers, March 1977); L. Raad and M. R. Thompson, *Discussion*, Transportation Research Record 733 (Transportation Research Board, 1979).

## Table A8

## ILLI-TRACK Input Data

### Loading Conditions

Wheel loading conditions representative of a 100-ton car were selected. A static wheel load of 32.9 kips (263-kips gross load for a 100-ton car) was used. The following area impact factor was applied.

$$\Theta = \frac{33 \times \text{Speed (mph)}}{100 \times \text{Wheel diameter (inches)}}$$

$\Theta$ = impact factor

Dynamic load = $(1 + \Theta)$ (static wheel load).

For the ILLI-TRACK study, the following conditions were used:

Static wheel load: 32.9 kips

Wheel diameter: 36 in.

Velocity: 50 mph

$\Theta$: 0.46

Dynamic wheel load: 48 kips

Dynamic wheel loadings were used in the ILLI-TRACK analyses. Wheel spacings were 70 in. center to center of axles and 40 in. from the end axle to the coupling line.

### Rail Information

132 RE

$I = 88.2\ \text{in.}^4$

$E = 30 \times 10^6$ psi

### Timber Ties

Width - 9 in.

Thickness - 7 in.

Length - 8.5 ft

Tie spacing - 20 in.

Compression modulus - 1250 ksi

Effective tie bearing length under each rail - 24 in.

## Table A8 (Cont'd)

**Ballast**

Area No. 4 Ballast

$E_R = 7000\ \Theta^{0.5}$

$E_R$ = Resilient modulus, psi

$\Theta = \sigma_1 + \sigma_2 + \sigma_3$, psi

Poisson's ratio = 0.35

Shear strength properties:

$C = 0$ $\phi = 40^o$

**Subgrade**

| | Medium subgrade | Soft subgrade | Very soft subgrade |
|---|---|---|---|
| Shear strength: | C = 11.5 psi, $\phi$ = 0 | C = 6.5 psi, $\phi$ = 0 | C = 3.1 psi $\phi$ = 0 |
| Poisson's ratio: | 0.45 | 0.45 | 0.45 |

| Repeated deviator stress, psi | Resilient modulus, psi | Repeated deviator stress, psi | Resilient modulus | Repeated deviator stress, psi | Resilient modulus, psi |
|---|---|---|---|---|---|
| 2.0 | 12,400 | 2.0 | 7,600 | 2.0 | 5700 |
| 6.2 | 7,500 | 6.2 | 3,000 | 6.2 | 1000 |
| 23.0 | 4,700 | 13.0 | 1,850 | 12.0 | 1000 |

## Table A9

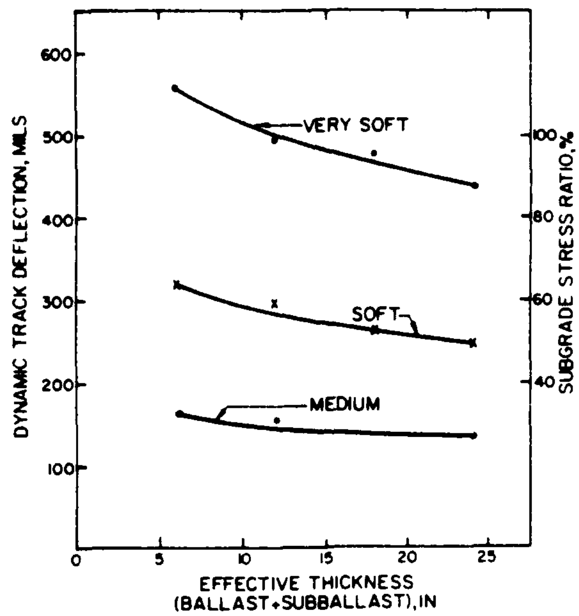
### ILLI-TRACK Response Summary

| Ballast thickness in. | Subgrade* | Max. tie reaction, kips | Tie mils $\Delta$, | Subgrade stress, psi $\sigma_1$ | $\sigma_3$ | $\sigma_D$ | Relative subgrade $\sigma$, %** |
|---|---|---|---|---|---|---|---|
| 12 | Medium | 18.5 | 153 | 26.7 | 16.9 | 9.8 | 43 |
| 18 | Medium | 20.7 | 138 | 24.3 | 15.4 | 8.9 | 39 |
| 24 | Medium | 22.4 | 133 | 22.1 | 14.2 | 7.9 | 34 |
| 12 | Soft | 16.6 | 299 | 23.6 | 16.6 | 7.0 | 54 |
| 18 | Soft | 19.3 | 266 | 21.7 | 15.3 | 6.4 | 49 |
| 24 | Soft | 21.5 | 248 | 20.0 | 14.2 | 5.8 | 45 |
| 12*** | Very Soft | 15.4 | 493 | ---- | ---- | ---- | 100 |
| 18 | Very Soft | 17.4 | 479 | 17.1 | 13.0 | 4.1 | 66 |
| 24 | Very Soft | 20.2 | 438 | 15.6 | 11.9 | 3.7 | 60 |
| 6 | Medium | 16.7 | 159 | 30.0 | 18.8 | 11.2 | 49 |
| 6 | Soft | 15.1 | 319 | 26.0 | 18.4 | 7.6 | 59 |
| 6 | Very Soft | 18.2 | 553 | 34.2 | 28.0 | 6.2 | 100 |

*Subgrade strengths: medium - $q_u$ = 23 psi; soft - $q_u$ = 13 psi; very soft - $q_u$ = 6.2 psi.

**Relative subgrade stress - 100 (subgrade stress/subgrade strength) = 100 ($\sigma_D/q_u$).

***Extensive subgrade and ballast failure. Stress data are not valid.




| Unconfined Strength (psi) | Subgrade Class |
|---|---|
| <10 | Very Soft |
| 10-20 | Soft |
| >20 | Medium |

**Figure A9. Effective thickness, dynamic track deflection, subgrade stress ratio relations (data from Table A9).**

A regression equation was developed that related relative subgrade stress and track deflection.

$$S = 19.2 + 0.127\ \Delta \qquad \text{[Eq A1]}$$

where:

S = relative subgrade stress, %

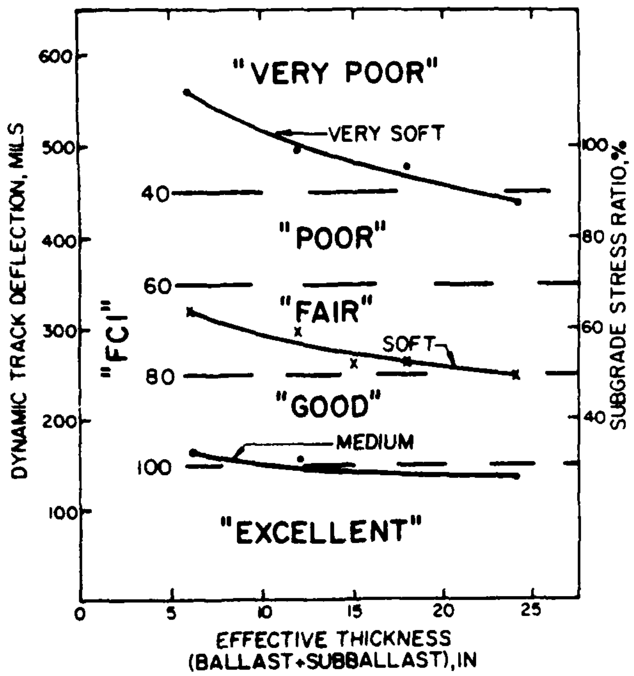
$\Delta$ = track deflection, mils

R = 0.89 standard error of estimate = 7.5.

Relative subgrade stress indicates general overall track strength and, more specifically, subgrade permanent deformation accumulation potential. Track performance assessment must also consider ballast and subballast permanent deformation accumulation potential.

Figure A10 shows a proposed FCI/Dynamic Track deflection rating. The data shown relate specifically to the database presented in Table A9.

The resilient behavior ($E_R = k\theta^n$ relation) of ballast/subballast-type granular materials can be estimated for general FCI rating purposes. Thus, the combined thickness of the ballast and subballast layers is the major factor influencing track deflection. For preliminary/general purposes, subgrade cohesive soils can be classified based on compressive strength. Table A10 gives a tentative classification.




| Unconfined Strength (psi) | Subgrade Class |
|---|---|
| <10 | Very Soft |
| 10-20 | Soft |
| >20 | Medium |

**Figure A10. Illustrative FCI rating relations (data from Table A9).**

**Table A10**

**Subgrade Rating**

| In-situ soil strength ($q_u$/psi) | Subrade rating |
|---|---|
| >30 | Stiff |
| 20-30 | Medium |
| 10-20 | Soft |
| <10 | Very soft |

Several data sources were used to develop a tentative FCI concept: mechanistic procedures and concepts (repeated load testing for resilient and permanent deformation behavior; ballast, subballast, subgrade soil shear strength; and ILLI-TRACK stress-dependent structural model with failure criteria).

To use the concept, the required input data are (1) total thickness of the granular material in the ballast/subballast layers and (2) subgrade unconfined compressive strength. Figure A10 is used to select the initial FCI. Although this value indicates track strength, it must be adjusted for track performance potential.

Deduct values for track performance are based on properties and characteristics that significantly influence permanent deformation (track settlement) behavior of granular materials. The deduct values are determined for the granular material in the thickness zone from the bottom of the tie to a 12-in. depth. Deduct values should be related to traffic density. The values are larger for increased traffic.

If the ballast or subballast granular materials meet "specification requirements," no deduct value is assigned. If the granular material deviates from "specification requirements," a deduct value is assigned. Examples of factors that would warrant deduct values are (1) gradation problems, particularly excess fines (-#4, -#200), (2) plastic fines, and (3) presence of "rounded and smooth" particles. Additional consideration is needed to establish appropriate "deduct values" for granular materials.

Any track section that shows "pumping" should be assigned a large deduct value.

The FCI rating chart shown in Figure A10 is for a given set of rail/tie/loading conditions. Other rating charts can be developed for a range of conditions (rail size, tie size and spacing [including missing ties], and unloading). ILLI-TRACK is the only current track structure model that can accommodate the wide range of conditions which must be considered.

## 7 SUMMARY

Foundation condition/track stability concepts have been developed that provide a comprehensive mechanistic framework (based on the ILLI-TRACK model) for considering track stability. Wheel-loading (magnitude-spacing), track parameters (rail/ties), ballast/subballast, and subgrade variables are considered in the procedure.

Dynamic track deflection is proposed as an initial track response for quantifying track stability. The validity of the track stability rating depends on the "precision" of the inputs.

APPENDIX B:

# DEVELOPMENT OF A RAILROAD TRACK GEOMETRY RATING SYSTEM*

## 1 INTRODUCTION AND BACKGROUND

The U.S. Army is responsible for maintaining about 3000 miles of railroad track in the continental United States. This track is distributed among 81 installations, with the largest having just over 200 miles. Daily traffic is generally light by mainline railroad standards; however, a number of installations would have important responsibilities during an emergency mobilization, which would produce higher traffic levels.

The FEs responsible for maintaining such a small amount of trackage do not normally have specialist railroad engineering skills. This, combined with the lack of suitable track inspection and maintenance manuals, means that maintenance may be spotty and expensive. Also, there may be cases of seriously deficient track at critical locations.

Accordingly, the U.S. Army Construction Engineering Research Laboratory (USA-CERL) has begun a program to develop a comprehensive maintenance management system for use on Army railroad track. The system will be based on the successful PAVER system[14] for maintaining highway and parking lot pavements developed previously by USA-CERL. The system will be designed for use by FEs who do not have specialist railroad engineering knowledge.

The first step in developing this system was to review the scope of the problem and assess the maintenance management techniques used in the commercial railroad industry. The results of this review are contained in USA-CERL Technical Report M-85/04. The principal conclusions of this study were:

1. Significant problems exist that are related to railroad track maintenance management at Army installations:

a. Lack of experienced personnel

b. Lack of funding, leading to a significant amount of deferred maintenance

c. Lack of information that would enable FEs to identify track deficiencies and set repair priorities

d. Lack of standards and procedures for educating personnel, identifying track defects, and gathering track inventory and condition data.

2. Maintenance management systems developed by the commercial railroad industry have generally been designed for high tonnage lines and are thus not appropriate for Army use. There are no maintenance management systems for short line railroads or industry trackage whose use is similar to that of the Army's trackage.

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*Prepared by J. A. Bing, Arthur D. Little, Inc., Cambridge, MA.

[14] M. Y. Shahin and S. D. Kohn, *Pavement Maintenance Management for Roads and Parking Lots.*

3. A maintenance management system tailored to the FEs' needs should be developed. It should include:

a. A uniform, objective, and inexpensive track inspection procedure. Inspection results should be used to determine a quantitative TCI.

b. A systematic database for storing and retrieving track inventory and condition information.

c. Guidelines and standards for M&R projects and for determining priorities.

d. Procedures for performing life-cycle project cost analyses of M&R projects.

These conclusions were supported by two independent studies: one on the nationwide problem of maintaining adequate rail services for defense needs and one for discussing track conditions at an individual base.[15]

Thus, USA-CERL elected to continue developing a railroad track maintenance management system. The next step was to develop inspection techniques for each major track component, which led to the development of condition indices for ballast and subgrade, ties, and track geometry.

This study describes the investigation of methods for assessing track geometry and for developing a Track Geometry Condition Index (TGCI). This investigation was made up of four steps:

Step 1. Defining railroad track functions.

Step 2. Determining ways to measure track geometry that relate both to the track's ability to perform its functions and to maintenance requirements.

Step 3. Defining the TGCI based on geometry measures.

Step 4. Evaluating track geometry measurement methods and assessing techniques for processing measurement data to obtain the measures defined in Step 2.

The following chapters of this report describe each investigation.

## 2 FUNCTIONS OF TRACK

The functions of railroad track are:[16]

1. Supporting the movement of traffic with an acceptably low risk of track-caused derailments

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[15]*Report to the Secretaries of Defense and Transportation: Federal Actions Needed to Retain Essential Defense Rail Service*, GAO/PLRD-83-73 (U.S. General Accounting Office, May 30, 1983); Gary A. Gordon, "Upgrading Rail Facilities at Fort Devens," *The Military Engineer*, No. 453 (November-December 1982).

[16]A. J. Bing and A. Gross, "The Future--Understanding Roadway Deterioration Modeling," *Symposium on the Uses of Track Geometry Car Information, American Railway Engineers Association Committee 32*, New Orleans (October 1982).

2. Permitting a quality of service (speed and ride quality) that meets the needs of railroad customers.

In this case, the "customer" is the Army itself, and has laid down a general standard that track should be at a minimum standard of Federal Railroad Administration (FRA) Class 2,[17] which permits freight operation at 25 mph.

Ride quality is generally important in passenger operations and sometimes for high-speed freight such as TOFC (Trailer on Flat Car) traffic. Since Army equipment and materials normally moved by rail are reasonably rugged in order to survive commercial railroad system conditions, ride quality is of minimal importance on Army-owned track.

This leaves derailment risk as the prime determinant of track condition. Track must be of a standard that minimizes derailment risk at the desired operating speed of 25 mph. Derailments must be avoided because:

1. They are costly.

2. They disrupt operations, leading to delay and additional expense. At worst, they could tie up a critical facility during a mobilization effort.

3. If hazardous materials are being moved, there is risk of a spill, fire, or explosion.

Thus, track geometry measurement procedures for Army use must be designed to detect and quantify track conditions likely to lead to derailment at speeds at and below the desired operating speed of 25 mph.

## 3 RELATIONSHIP BETWEEN TRACK GEOMETRY AND TRACK FUNCTION

Four basic geometry measurements are required to define the actual position of both rails of a railroad track relative to their "correct" position (Figure B1). These are crosslevel, profile, alignment, and gauge.

Numerous measures of track geometry can be derived arithmetically from these basic measurements. Examples of derived measures are:

1. Twist or warp, which is the difference in crosslevel between two points that are a specified distance apart.

2. Mid-chord-offsets (MCO), alignment, or profile measured at the midpoint of a string stretched between two points on the rail (Figure B2).

Derailment risk is generally related to some type of derived track geometry measure. To determine the causes of track-geometry-related derailments, the statistics of railroad derailments and current research into derailment mechanisms were reviewed.

Study of track-caused railroad accidents reported to the FRA by commercial railroads indicates that most such accidents occur at relatively low speed on low-class track similar to the type found at Army installations. This, and the types of track

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[17] *Code of Federal Regulations*, Title 49, Part 213, "Track Safety Standards."

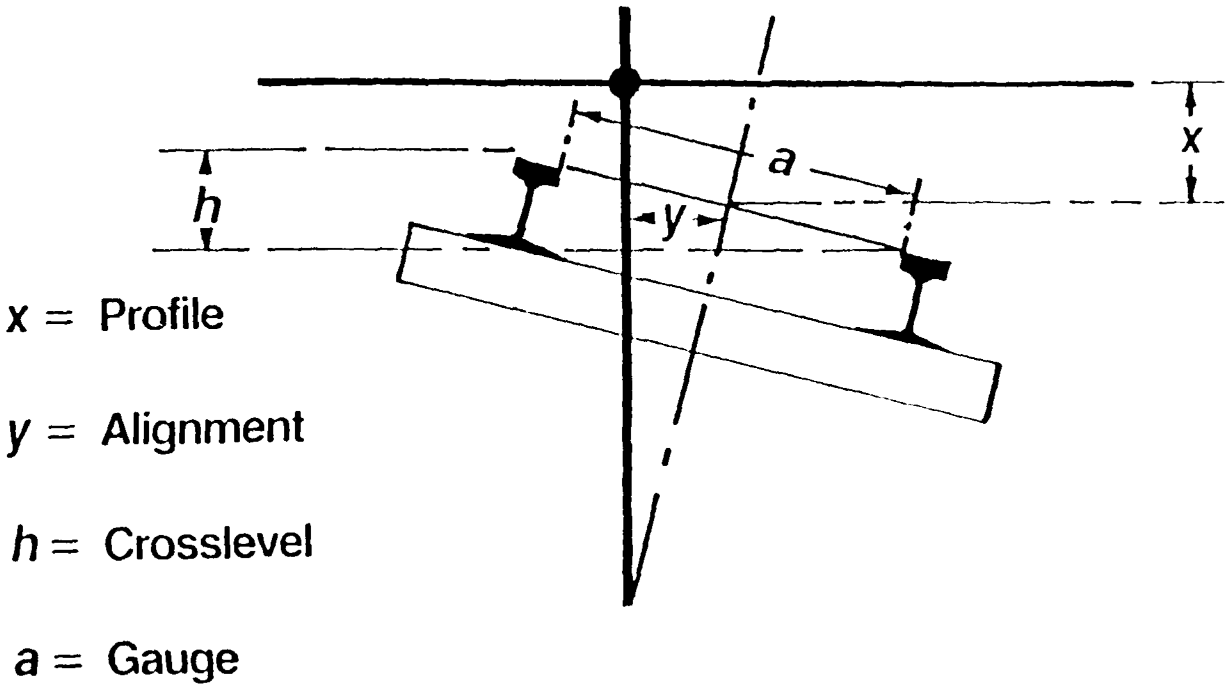


Figure B1. Track geometry measurements.

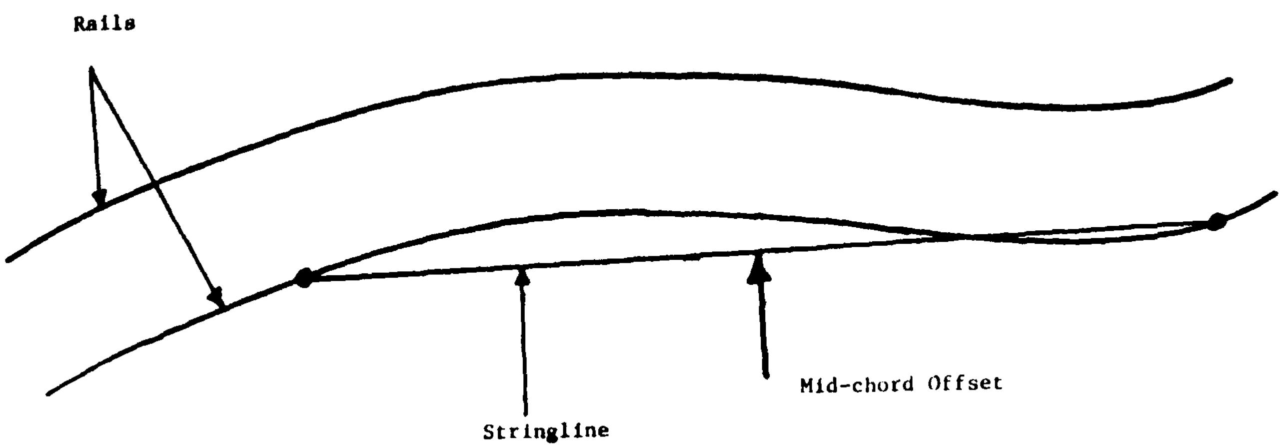


Figure B2. Example of a mid-chord offset.

defects that lead to derailments are illustrated by accident statistics[18] in a typical recent year. There were 2273 reported track-caused accidents in 1981, producing reported damage of $80 million. Most of these occurred at low speed, as shown in Table B1.

From Table B1, it can be deduced that 94 percent of accidents, representing 72 percent of reported cost, occurred at speeds below 30 mph--typical of Army tracks. Therefore, the reported causes of commercial railroad track-caused accidents can be expected to be a good guide to the type of accidents that can be expected on Army track.

Table B2 lists the number and average cost of track-caused accidents, broken down by reported cause. Generally the causes having a high reported per accident cost will be those that occur on high-speed main-line track; those with a low to moderate cost will be more common on low-speed running tracks and yard tracks.

Most accidents caused by poor alignment resulted from the track buckling in hot weather. Since this type of incident usually occurs on welded rail track, it is unlikely to occur on Army tracks, which have little or no welded rail. Other "high-cost" accidents are those caused by signal system failure, joint bar defects, and incorrect superelevation. The rest of the causes of accidents are likely to be characteristic of those to be expected on Army tracks.

The types of common track defects likely to be detected by track geometry measurements are:

1. Excessive gauge

2. Poor crosslevel

3. Some defects at track appliances, where these exhibit poor geometry

4. Some roadbed defects where these exhibit poor geometry.

The next step is to determine how to use track geometry measurements to detect potentially dangerous track conditions and to determine maintenance needs. This can be done by reviewing the types of component deterioration that lead to a maintenance requirement, and the mechanism of the derailment that occurs if deterioration is allowed to go too far.

The three major components of track are the rail, the ties, and the ballast and roadbed.

Rails deteriorate either by growth of fatigue flaws, side wear of the head in curves, and wear and crushing of the top of the head. Only side wear in curves can be detected by track geometry measurements, since it produces wide gauge.

Ties deteriorate by mechanical damage from train loading and by timber decay. Because traffic is light on Army tracks, timber decay will be the predominant form of tie deterioration. The decayed timber will fail to support the rail laterally and vertically,

[18] *Accident/Incident Bulletin, No. 150, Calendar Year 1981* (U.S. Department of Transportation, FRA, Office of Safety, June 1982).

## Table B1

### Track-Caused Accidents by Speed - 1981

(From *Accident/Incident Bulletin, No. 150, Calendar Year 1981* [U.S. Department of Transportation, FRA, Office of Safety, June 1982].)

| Speed Range (mph) | Main Line | Yard Tracks | Industry Tracks + Any Other | Total | Aggregate Reported Cost $1000s | Cost Per Accident, $ |
|---|---|---|---|---|---|---|
| 0- 10 | 447 | 1036 | 242 | 1725 | 28,075 | 16,275 |
| 11- 20 | 167 | 19 | 22 | 208 | 11,039 | 53,072 |
| 21- 30 | 198 | 1 | 3 | 202 | 18,525 | 91,708 |
| 31- 40 | 75 | 1 | 0 | 76 | 12,152 | 159,895 |
| 41- 50 | 29 | 2 | 0 | 31 | 5,215 | 168,226 |
| 51- 60 | 11 | 0 | 0 | 11 | 1,859 | 107,809 |
| Over 60 | 11 | 0 | 0 | 11 | 3,432 | 312,000 |
| Unknown | 2 | 7 | 0 | 9 | 81 | 9,000 |
| Totals | 940 | 1066 | 267 | 2273 | 80,378 | 35,362 |

## Table B2

### Track-Caused Accidents by Type of Defect

(From *Accident/Incident Bulletin, No. 150, Calendar Year 1981* [U.S. Department of Transportation, FRA, Office of Safety, June 1982].)

| Reported defect | Number of Accidents | Cost per accident, $ |
|---|---|---|
| Roadbed defects, washout, settlement, etc. | 111 | 34,052 |
| Excessive gauge | 505 | 15,940 |
| Poor alignment | 159 | 70,390 |
| Poor profile | 8 | 18,500 |
| Incorrect superelevation | 33 | 80,820 |
| Poor crosslevel | 309 | 32,300 |
| Other track geometry defects | 34 | 40,030 |
| Rail defects | 519 | 56,150 |
| Joint-bar defects | 52 | 65,380 |
| Switches, frogs, and track appliances | 507 | 19,330 |
| Other way and structures | 24 | 38,000 |
| Signal system failures | 12 | 226,670 |

leading to profile, crosslevel, and gauge track geometry deviations. Of these, gauge deviations are the most important, since they are most likely to cause accidents.

Ballast and roadbed deteriorate by progressive breakdown and clogging with fine material and vegetation. This impedes drainage, reduces load-bearing capacity, and produces significant profile and crosslevel track geometry variations. An important "special case" of ballast degradation is the periodic local settlement under rail joints; track with poor ballast, old rail, and possibly loose or worn joint bars will also exhibit this condition. With standard North American 39-ft, staggered-joint rail, a series of low joints produces a resonant response in high center-of-gravity freight cars at 15 to 20 mph, leading to "rock and roll" derailments.

Thus, it is logical that the following two derailment mechanisms are associated with track geometry defects:

1. With greatly excessive gauge, the wheelset simply drops between the rails; however, this is a rare situation. More commonly, gauge-related derailments are a combination of wide gauge and weak rail-tie fastening, permitting the rail to be displaced laterally or to roll over under vehicle loads. Alternatively, the leading wheelset of the tracks can generate a high angle of attack with respect to the rail, causing the flange to climb over the rail. The situation is aggravated by crosslevel variations that reduce wheel load and curvature.

2. Two mechanisms are associated with crosslevel variations. Excessive track twist can cause unloading of a car's wheels on diagonally opposite sides of the tracks. This situation is worst when cars with a stiff body structure, such as a tank car, are unladen. The second mechanism is the rhythmic rock and roll of high-center-of-gravity cars at 15 to 20 mph. A series of staggered low joints causes this problem.

Two of the four basic track geometry measurements--crosslevel and gauge--thus provide the most significant indicators of maintenance needs and derailment risk. (Table B3 summarizes the relationships.) Therefore, it is not essential to measure the other two

**Table B3**

**Relationship Between Track Geometry Parameters, Maintenance Needs, and Derailment Mechanisms**

| Track geometry parameter | Track component degradation | Derailment mechanism |
|---|---|---|
| Crosslevel | • Ballast settlement and degradation, particularly at rail joints<br>• Old "surface bent" rail<br>• Loose/worn joint bars | • Rock and roll at 15 to 20 mph<br>• Excessive twist, leading to wheel unloading |
| Gauge | • Decayed ties<br>• Poor rail/tie fastenings<br>• Curve-worn rail | • "Drop in" between rails<br>• Rail lateral displacement or rollover<br>• Flange-climbing |

parameters (profile and alignment). There is strong correlation between crosslevel and profile, as noted by both an Arthur D. Little, Inc., analysis for the FRA and preceding work by ENSCO, Inc.[19] Poor profile generally accompanies poor crosslevel, and there is little evidence of accidents caused by poor profile.

Alignment is less easy to dismiss, and information on it should probably be regarded as "nice to have." Poor alignment is not reported as a prime cause of derailments, but in many cases is probably an important contributing factor. This is supported by comparing derailment records and track geometry measurements made by the Southern Railway (SR). The SR found that for main-line track (normally FRA Class 3 or Class 4), accident incidence correlated with alignment.[20] However, the basic causes of poor alignment are a combination of tie and ballast deterioration, which are already indicated by the gauge and crosslevel measurements. Also, alignment is more difficult and costly to measure than gauge or crosslevel.

## 4 TRACK GEOMETRY ANALYSIS AND TRACK GEOMETRY CONDITION INDEX

### Introduction

The previous section showed that gauge and crosslevel are the most important geometry parameters, and developed a qualitative description of how the parameters relate to derailment mechanisms and track component deterioration. The next step is to define how to analyze track geometry data to obtain descriptors that quantify track condition and potential to cause derailments. The values of these descriptors for a specific section of track can be summed to arrive at a numerical Track Geometry Condition Index (TGCI).

The TGCI must provide a measure of track quality from two points of view: (1) an overall condition measure that is indicative of maintenance needs and (2) a derailment risk measure. These two measures differ in that a specific track section could have a small number of severe defects that would mean high derailment risk, but that would not necessarily be costly to repair. Therefore, two separate condition indices are developed. The following discussions refer to calculation of condition indices for a specific track section.

### Maintenance Condition Index—TGCI(M)

The maintenance condition index should be derived from statistics of all geometry measurements in the section.

Crosslevel: Finding the root mean square of all measurements in the section is recommended. Based on FRA research, the numerical value can vary from 0.05 in. on high-quality passenger track to 0.3 in. on track that is approaching minimum FRA Class 2 standards. This measure will relate to ballast and roadbed degradation.

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[19] *Accident/Incident Bulletin, No. 150, Calendar Year 1981*; A. J. Bing, *Development of a Track Degradation Modeling Technique*, FRA Report No. DOT/FRA/ORD-83/12 (Department of Transportation, 1983).

[20] R. F. Tuve and R. G. Thomas, *Direct Application of Track Geometry Planning and Derailment Reduction* (Southern Railway System, 1980).

Gauge: Finding the mean and standard deviation of all gauge measurements in the section is recommended as joint measures of gauge. A high average gauge indicates curve-worn rail and/or numerous bad ties; a high gauge standard deviation indicates patches of bad ties.

Numerical values of mean gauge vary between 56.7 in. on good track to 57.25 in. on marginal Class 2 track. Numerical values of gauge standard deviation vary between 0.05 in. on good track to 0.3 in. on marginal Class 2 track.

The preferred system for calculating the TGCI is to give perfect track an index of 100 and to deduct from it an amount indicative of the deterioration. For initial trials of the TGCI(M), equal weight is given to each of the three measures, and each is scored on a range of 0 ("perfect" track) to 20 (typical of marginal Class 2 track). This gives approximately double weight to the gauge measures, as compared to the crosslevel measures. This is appropriate, since it is more costly to correct gauge problems (which require new ties, refastening of the rail, and sometimes new rail) than to correct a crosslevel problem by adding ballast and surfacing. This is similar to the weighting used by the Norfolk and Western Railroad in their track geometry rating system.[21]

Thus, the formula for the "maintenance index," TGCI(M) is:

$$\begin{aligned} 100 &- 70\ (\text{Crosslevel r.m.s.}) \\ &- 25\ (\text{gauge} - 56.5) \\ &- 70\ (\text{gauge standard deviation}) \end{aligned}$$

The multipliers have been rounded off to convenient whole numbers, and all measurements are in inches. Using this formula, one would expect Class 2 track to have a TGCI(M) between 40 and 60.

An alternative to crosslevel root-mean-square (if simple to measure) is the root-mean-square of track twist over 20 ft. These two measures are highly correlated, and one can be substituted for the other by adjusting the multiplier appropriately. Generally, the warp measurements are about 75 percent higher than the crosslevel standard deviation on the same piece of track. The multiplier should therefore be adjusted downward from 70 to 40 to give the same weight to this parameter. The choice between the crosslevel and warp measurement can be based simply on convenience of measurement.

**Safety Track Geometry Condition Index--TGCI(S)**

The safety index is derived by analyzing exceptions to acceptable standards. The standards of acceptability suggested are based on FRA standards for Class 1 and Class 2 track and on research by the Transportation Systems Center of the Department of Transportation into the freight car "rock and roll" phenomenon. Table B4 lists the suggested standards.

Aubrey E. Harvey, *Applying Statistical Quality Control Methodology to the Design of a Track Quality Index* (Norfolk and Western Railway, 1982).

Table B4

Track Geometry Safety Thresholds

| Track geometry measures/derailment mechanism | Track class and speed | |
|---|---|---|
| | **FRA Class 2**<br>**25 mph** | **FRA Class 1**<br>**0 mph** |
| Crosslevel - Rock and Roll | Root Mean Square Deviation over any 400 ft of 100 ft running mean removed cross-level shall not exceed 0.35 in modified (TSC rock and roll standard) | Not applicable |
| Track Twist - Wheel Unloading | Difference in crosslevel must not exceed 2 in. between any two points less than 62 ft apart. | Difference in cross-level must not exceed 3 in. between any two points less than 62 ft apart. |
| Gauge - Wheel Climb, "Drop In," Rail Development, or Rollover | Gauge must be between 56 and 57-1/2 in. (57-3/4 in. on curves of more than 2 degrees) | Gauge must be between 56 and 57-3/4 in. |

Deduct values for exceeding the safety standards are designed to fall in the range of 60 to 100, clearly distinguishing them from typical values of the "maintenance" index.

The deduct values have been worked out to give the following general results:

1. Less than 60; no violations of Class 2 standards.

2. Between 60 and 70:

   a. No violations of Class 1 standards

   b. Limited violations of Class 2 standards; 25 mphoperation is permitted after on-the-ground inspection of defects, and under direct supervision of the FE's staff.

3. Between 70 and 80:

   a. No exceedences of Class 1 standards.

   b. Numerous violations of Class 2 standards; maximum speed of operation 10 mph.

4. Between 80 and 90: limited exceedences of Class 1 standards. Operations permitted after on-the-ground inspection of defects and under direct supervision of the FE's staff.

5. Between 90 and 100: numerous exceedences of Class 2 standards; no operations are permitted until the track is repaired.

Because of the way the measures are defined, exceedences of the rock and roll and the gauge standards can occur over a whole section of track; however, it is physically impossible for the twist standard to be exceeded over more than about 10 to 15 percent of the track.

Based on a review of actual exceedences measured on sample track, the preliminary deduct value formulas given below are suggested. Since the data available for review were generally for track at FRA Class 3, with only infrequent exceedences, the coefficients are estimates only and subject to modification based on results achieved in a pilot test of Class 1 or Class 2 track.

Deduct value is the larger of the values given by the following formulas, provided exceedences to the standards are present.

Class 2 Formula:

TGCI(S) = 60 + .05 (% track in section exceeding rock and roll index of 0.35 in.)
+ 5 (% of track exceeding FRA Class 2 twist standard)
+ % of track exceeding FRA Class 2 gauge standard

but not to exceed 79

or

Class 1 Formula:

TGCI(S) = 80 + 5 (% of track exceeding FRA Class 1 twist standard)
+ % of track exceeding FRA Class 1 gauge standard

but not to exceed 100.

If a single-number TGCI is required, this could be the higher of the maintenance or safety indices. However, both numbers are needed if the user wishes to evaluate both safety of operation and maintenance requirements.

Use of these formulas assumes the availability of some means of making continuous crosslevel and gauge measurements and of automatic processing of the measurements. This availability is both feasible and desirable. Section 5 of this study discusses measurement and analysis equipment.

### Sample Deduct Value Calculations

Some sample deduct value calculations are provided below to illustrate use of the formulas. The only data available are for main-line railroad track generally maintained to FRA Class 3 or 4. This means that the track does not violate FRA Class 1 or 2 standards. Therefore, the sample calculations for the deduct values are carried out with real data for the "maintenance" index TGCI(M), but hypothetical data for the "safety" index TGCI(S).

Table B5 gives the TGCI(M) calculation for five segments that vary in length between 1/2 and 1 mile. The figures for segments 1 through 4 indicate track that is of moderate quality, marginally FRA Class 3 standard, and only just adequate for its posted speed of 50 mph. Segment 5 is in a city with numerous grade crossings and turnouts, and has a posted speed of 35 mph. Again, quality of this segment is barely adequate. This track was maintained in the year following the measurements.

Table B6 gives some hypothetical calculations of the safety index TGCI(S). Segments 1 and 2 are typical of track between Class 2 and Class 1 quality, and segments 3 and 4 are characteristic of track that is below Class 1. The relatively high values for the percent exceedence of the rock and roll index are realistic. Since this index is calculated over a 400-ft window, it is likely that areas of exceedence will cover a large proportion of a track segment.

## 5 TRACK GEOMETRY MEASUREMENT AND DATA ANALYSIS

Track geometry measurement systems used in the railroad industry vary from the heavyweight "full-service" track geometry car with on-board data processing, down to

Table B5

Sample Maintenance Index Calculations

| | Segment numbers | | | | |
|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 |
| **Raw measurement data** | | | | | |
| Crosslevel rms (in.) | 0.170 | 0.238 | 0.327 | 0.242 | 0.448 |
| Mean gauge (in.) | 57.20 | 57.26 | 56.92 | 57.02 | 57.23 |
| Gauge standard deviation (in.) | 0.13 | 0.15 | 0.13 | 0.14 | 0.20 |
| **Deduct values** | | | | | |
| Crosslevel | 11.9 | 16.7 | 22.3 | 16.9 | 31.4 |
| Mean gauge | 17.5 | 19.0 | 10.5 | 13.0 | 18.3 |
| Gauge standard deviation | 9.1 | 10.5 | 9.1 | 9.8 | 14.0 |
| Total deduct value | 38.5 | 46.5 | 41.9 | 39.7 | 63.7 |
| TGCI(M) | 61.5 | 53.8 | 58.1 | 60.3 | 36.3 |

Table B6

Hypothetical Safety Index Calculations

| | Segment numbers | | | |
|---|---|---|---|---|
| | 1 | 2 | 3 | 4 |
| **Raw measurement data** | | | | |
| % Exceedence rock and roll index | 30 | 50 | 70 | 90 |
| % Exceedence 2-in. warp over 62 ft | 1 | 2 | 3 | 4 |
| % Exceedence 3-in. warp over 62 ft | 0 | 0 | 1 | 2 |
| % Exceedence gauge of 57-1/2 in. | 2 | 4 | 6 | 8 |
| % Exceedence gauge of 57-3/4 in. | 0 | 1 | 3 | 5 |
| Class 2 deduct value | 69.5 | 76.5 | 79 | 79 |
| Class 2 TGCI(S) | 30.5 | 23.5 | 21 | 21 |
| Class 1 deduct value | N/A | 81 | 88 | 95 |
| Class 1 TGCI(S) | N/A | 19 | 12 | 5 |

the traditional crosslevel, gauge bar, and stringline manual techniques. Each of the common systems is briefly described below:

### Heavyweight Track Geometry Car

Heavyweight track geometry cars are used by the FRA Office of Safety and by many major railroads, including the Southern, Santa Fe, and Chessie system. The cars are usually full-sized passenger cars ballasted up to the weight of a loaded freight car (i.e., 200,000 to 250,000 lb). All track geometry parameters are measured using electronic sensors and the data fed into the data logging and processing equipment. The major advantage of this type of car is that the track is measured under load. The disadvantage is the cost of acquisition and operation. Cars of this type cost more than $1 million and are normally locomotive-hauled. Since they are rail-bound, movement between Army installations would have to be over the commercial rail system. This cost would therefore have to be added to that of providing locomotives and train crews at each Army installation surveyed. Thus, because of its operational difficulties and high cost, the heavyweight track geometry car is not suitable for surveying Army tracks.

### Lightweight, Self-Propelled Rail-Bound Track Geometry Car

This car is typified by the commercially produced Plasser EM80 car. It has many of the disadvantages noted for the heavyweight car, except that a locomotive is not required; also, the advantage of measuring track under load is lost. Thus, this type of car is also not suitable for surveying Army tracks.

### High Rail Track Geometry Car

These are typified by the Plasser EM25 car. All track geometry parameters are measured, and on-board data processing can be provided. Being relatively lightweight, it does not measure track under representative load. Its cost is about $500,000. Except for its expense, which is probably more than the Army wants to incur, this type of car is a feasible option. A single car could survey all Army track in the Continental United States in 1 year, traveling between installations by road.

### Journal Box Mounted Instrumentation

Two systems that use instrumentation mounted on the journal box of a rail vehicle to measure geometry have been developed in FRA research programs.

1. The gyroscope crosslevel index monitor measures crosslevel only, and an associated microprocessor calculates the "rock and roll" crosslevel index. One prototype has been manufactured; its unit production cost is estimated at $30,000 to $50,000.[22]

2. The Locomotive Track Hazard Detector consists of journal box mounted accelerometers and can measure profile, crosslevel, and alignment. "Off-line" data processing was used in the experiments, but the instrumentation could, in concept, be

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[22] *Instructional Manual for the Harmonic Crosslevel Monitor System Northrop Crosslevel Monitor System* (Northrop Precision Products Division, January 1982) (for Transportation Systems Center).

linked to a microprocessor to produce specific track quality measures. Minimum speed of operation is about 15 mph. No price is available.[23]

The advantages of both systems are that track is measured under load and that measurements can be made during normal train operations. Disadvantages are that additional equipment would be needed to measure gauge and that locomotives are not necessarily available at Army installations. Also, neither device is commercially available.

**Simple Lightweight Gauge and Crosslevel Measurement Systems**

This system is typified by the Nordberg "Track Inspector." Gauge and crosslevel are measured by a simple trolley towed behind any "high-rail" vehicle. In the standard device, measurement output is displayed on a readout dial, but the Nordberg Company has developed recording equipment to write the data on tape for off-line analysis. In principle, a microprocessor could be added to calculate selected geometry statistics for a track section "on-line," adapting the approach used in the FRA crosslevel index monitor.

The advantages of this approach are its low cost ($10,000 to $15,000 for the basic system without recording equipment) and its portability. Its main disadvantage is that the track is not measured under load.

A review of all these measurement systems indicates that a lightweight system such as the Nordberg "Track Inspector" is the most suitable for the Army, mainly because of its low cost and because it measures the parameters of most interest. Its use would be valuable to Army FEs, even without data analysis facilities needed to calculate deduct values. The enhancements needed to implement the full system as suggested are:

1. Development of programs for either microprocessor or personal computer to produce track-quality measures and deduct values from raw measurements.

2. For Army installations that have their own switching locomotives, investigation into attaching the Nordberg measurement axle to the locomotive truck near an axle. It may then be possible to measure track under load.

[23] J. Corbin, et al., *Locomotive Track Hazard Detector Program, Interim Report* (FRA, Office of Research and Development, August 1981).

## APPENDIX C:

## RAILROAD TIE CONDITION INDEX--A PRELIMINARY CONCEPT*

### 1 INTRODUCTION

The traditional way of determining tie condition is to count the number of bad ties in a given length of track, either through 100 percent sampling or a sampling of a fixed number (usually 100 ties per mile). There are several significant problems with this approach:

1. Sampling 100 ties/mile, usually always at the same location, is statistically not very accurate.

2. One hundred percent sampling is very time-consuming if a large amount of track is involved.

3. What is often considered a bad tie in heavy-density, high-speed track might be satisfactory in low-density, low-speed track. For example, a split tie that may not hold a spike will still distribute much of the loading. If this tie is located between two good ties, it would be quite satisfactory for low-density, low-speed track.

4. A straightforward tie count does not consider the number of clusters of bad ties. A single bad tie or a cluster of two bad ties is not considered a problem in low-speed, low-tonnage track. However, clusters of more than, say, four bad ties can be quite serious, constituting a potential derailment site. The number and size of clusters depend on the proportion of bad ties and whether ties have been spotted in to break up the clusters.

The purpose of this project was to develop a standardized method of rating track condition that is more accurate than tie counting and that considers the problems listed above.

### 2 DEFINITION OF A BAD TIE

Fundamental to developing a Tie Condition Index (TCI) is defining a bad tie; this definition depends on many factors, such as line speed, traffic density, percent of hazardous material carried, and maintenance policy.

The traffic pattern on a heavily used line might be such that it is inadvisable to have any bad ties in the track; thus, a bad tie would be defined as one that will not last until the next maintenance cycle. However, on a very lightly used line where a number of bad or marginal ties are acceptable, a bad tie may be one that is totally decayed. A marginal tie in the middle of a cluster of three would be a logical choice to change rather than one of the other two which, if changed, would still leave a cluster of two.

*Prepared by David R. Burns, Associate Consultant, W. G. Richmond & Co., Wilmette, IL.

To develop a TCI, it has been assumed that its use would be primarily on track where a tie will be removed only when it exhibits one or more of the properties defined in the FRA safety standards for a bad tie:

1. Broken through

2. Split or otherwise impaired to the extent that crossties allow the ballast to work through or do not hold spikes or rail fasteners

3. So deteriorated that the tie plate or rail base can move laterally more than 1/2 in. relative to the crossties

4. Cut by the tie plate through more than 40 percent of a tie's thickness.

Figure C1 illustrates typical bad ties.

## 3 DETERMINING TIE CONDITION INDEX

To determine Tie Condition Index, first select the accuracy level required.

1. Standard: This or any simplified method can be used with acceptable accuracy if there are fewer than 10 percent bad ties and/or the ties are primarily spike-killed or broken.

2. Improved: This method should be used if a significant proportion of the bad ties is either decayed or split and there are more than 10 percent bad ties.

To determine the sampling plan to be used:

1. Separate determinations should be made for tangents vs. curves greater than 2 degrees and for track exhibiting different drainage and/or subgrade characteristics. Since there are economies of scale on larger sample sizes, tangents interspersed with curves should be aggregated where possible.

2. Inspection should be in groups of 50 consecutive ties. These ties should be inspected by beginning the first group at a random location, near the start of the segment, and interspersing the remaining groups at roughly equal intervals corresponding to about 0.8 x segment length/(number of clusters + 1). Sample spacing can be referenced to pole lines, but sampling should start at a random distance from the pole.

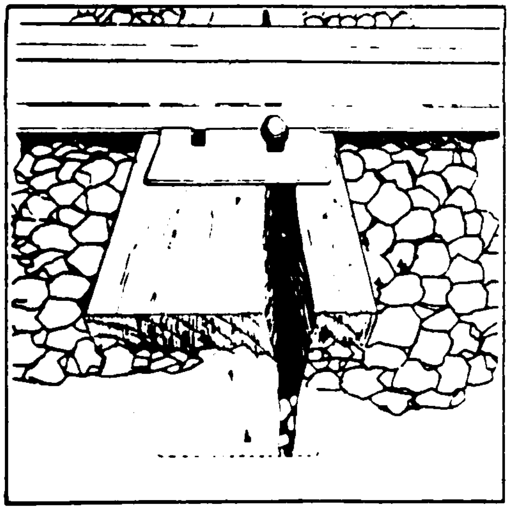
3. A minimum of six groups of 50 ties should be counted in any aggregate track segment.

4. Extra groups should be counted if the inspector notices unusually large variation in the proportion of failed ties in different groups within the same track segment.

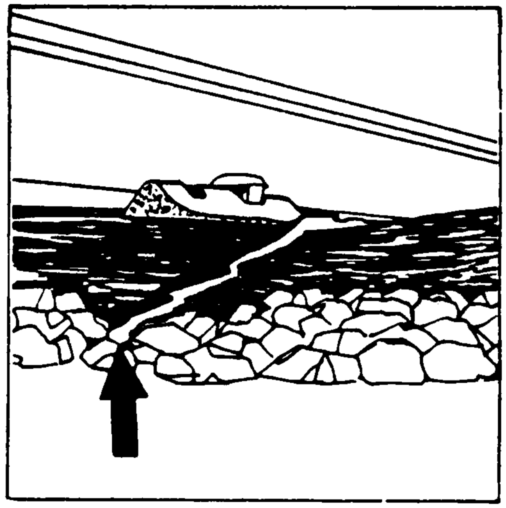
5. If the track segment length is less than 1/4 mile, the required groups can be reduced to four.

### Inspecting the Track

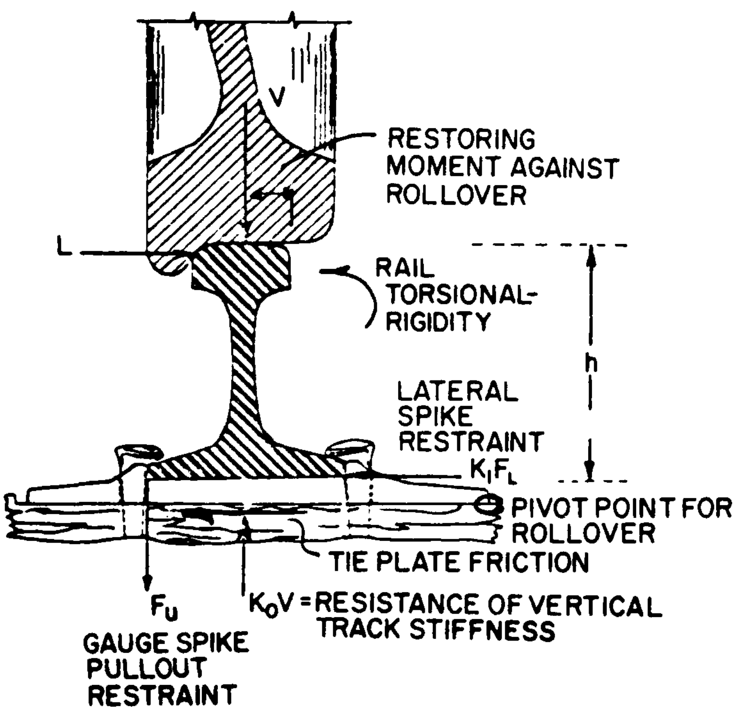
Using the sampling plan outlined above and the selected tie condition calculation form (Figure C2), the track should be inspected and the number of bad ties in each



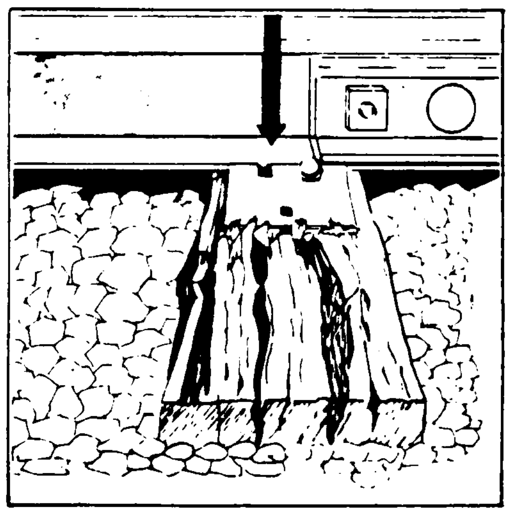
TIE SPLIT PROGRESSED THROUGH FIELD SIDE SPIKE



FLEXURAL BREAK THROUGH RAIL SEAT AREA




TIE REACTION FORCES



TIE DECAY WITH SPIKE KILL

**Figure C1. Typical bad ties.**

LOCATION A LENGTH 1 MILES SAMPLES REQUIRED 6 PAST MAINTENANCE; LAST MAJOR TIE REPLACEMENT PROGRAM 6 YEARS

TIES/MILE 2250 SPOT TIE REPLACEMENT YES NO

| 1 | 2 TO 11 | | | | | | | | | | 12 | 13 | 14 | 15 | 16 | 17 | 18 | 19 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SEG. NO. | BAD TIE COUNT FROM SAMPLES (a) | (b) | (c) | (d) | (e) | (f) | (g) | (h) | (i) | (j) | TOTAL $\Sigma$(a)-(j) (T) | PROPORTION BAD TIES (T)/Total Sample=(P) | BAD TIES/ MILE (N)x(P)=(BT) | ADJUSTED BAD TIES/ (BT)x.8=(Na) | BAD TIE LIMITATION FACTOR | PRELIM. INDEX (1-(Na)/(LF)) x100 = I | MAINT. FACTOR FROM TABLE C1 (MF) | TIE CONDITION INDEX |
| 1 | 5 | 8 | 12 | 5 | 15 | 17 | | | | | 62 | .20 | 671 | 537 | 2330 | 77 | 1 | 77 |
| 2 | 6 | 9 | 12 | 15 | 15 | 18 | | | | | 75 | .25 | 812 | 650 | 2230 | 72 | 1 | 72 |

Figure C2. Tie Condition Index calculation forms—standard accuracy.

sample noted in the correct location on the form. Criteria for a bad tie should be as listed under the definition of bad ties.

If improved accuracy is required, the type of defects in each sample must be noted (use the form shown in Figure C3). Data relative to length of segment, ties per mile, and tie replacement policy should also be noted.

**Calculating TCI Standard Accuracy** (See Figure C3)

After filling the number of bad ties in each sample for each segment, the following procedure should be followed:

1. The number of bad ties should be totaled and entered in Column 12.

2. The proportion of bad ties must now be calculated (i.e., the total bad ties divided by the total ties sampled).

3. To determine bad ties per mile (Column 14), multiply the proportion calculated in Column 13 by the number of ties per mile.

4. To allow for the fact that some of the bad ties are supporting the track or holding gauge, the number of bad ties per mile should be multiplied by 0.8 to give adjusted bad tie count (Column 15).

5. The bad tie limitation factor (LF) can be calculated from either

$$\frac{63360}{\text{tie spacing in inches}} - 920\text{, or ties per mile} - 920.$$

6. Preliminary index is calculated from the formula shown in Column 17.

7. The maintenance factor (MF) to allow for the number of clusters and the maintenance policy can be determined from Figure C4. Reading up from the proportion of bad ties to the appropriate maintenance cycle gives the MF on the y-axis.

8. The TCI is the preliminary index (Column 17) multiplied by the MF (Column 18).

**Calculating TCI Improved Accuracy** (See Figure C3)

1. The number of bad ties should be totaled and entered in Column 12.

2. The proportion of bad ties can now be calculated (i.e., the total bad ties divided by the total ties in the sample).

3. To determine bad ties per mile (Column 14), multiply the proportion calculated in Column 13 by the number of ties per mile.

4. To allow for the fact that some of the bad ties are supporting the track or holding gauge, the number of bad ties per mile should be multiplied by the respective adjustment factor to determine the adjusted bad tie count that is totaled in Column 16.

LOCATION A LENGTH 1 MILES SAMPLES REQUIRED 6

TIES/MILE 3250

PAST MAINTENANCE; LAST MAJOR TIE REPLACEMENT PROGRAM 6 YEARS

SPOT TIE REPLACEMENT YES/NO

| | 1 | 2 TO 11 | | | | | | | | 12 | 13 | 14 | 15 | 16 | | 17 | 18 | 19 | 20 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| SEG. NO. | DEFECT TYPE | BAD TIE COUNT FROM SAMPLES (a) | (b) | (c) | (d) | (e) | (f) | (g) | (h) | TOTAL $\Sigma$(a)-(h) (T) | PROPORTION BAD TIES (T)/Total Sample=(P) | BAD TIES/ MILE (N)x(P)=(BT) | ADJUST. FACTOR (AF) | ADJUSTED BAD TIES/ (BT)x(AF)=(Na) | TOTAL | BAD TIE LIMITATION FACTOR | PRELIM. INDEX (1-(Na)/(LF)) x100 = I | MAINT. FACTOR FROM TABLE C1 (MF) | TIE CONDITION INDEX |
| 1 | DECAY | 3 | 4 | 7 | 6 | 5 | 2 | | | 27 | .09 | 293 | 0.90 | 264 | | | | | |
| | SPLIT | 1 | 1 | 2 | 3 | 5 | 5 | | | 17 | .06 | 195 | 0.67 | 132 | 636 | 2330 | 73 | 1 | 73 |
| | SPIKE | 1 | 1 | 0 | 2 | 3 | 1 | | | 8 | .03 | 98 | 0.84 | 82 | | | | | |
| | BROKEN | 5 | 3 | 5 | 1 | 1 | 2 | | | 17 | .06 | 195 | 0.81 | 158 | | | | | |
| | DECAY | | | | | | | | | | | | 0.90 | | | | | | |
| | SPLIT | | | | | | | | | | | | 0.67 | | | | | | |
| | SPIKE | | | | | | | | | | | | 0.84 | | | | | | |
| | BROKEN | | | | | | | | | | | | 0.81 | | | | | | |
| | DECAY | | | | | | | | | | | | 0.90 | | | | | | |
| | SPLIT | | | | | | | | | | | | 0.67 | | | | | | |
| | SPIKE | | | | | | | | | | | | 0.84 | | | | | | |
| | BROKEN | | | | | | | | | | | | 0.81 | | | | | | |
| | DECAY | | | | | | | | | | | | 0.90 | | | | | | |
| | SPLIT | | | | | | | | | | | | 0.67 | | | | | | |
| | SPIKE | | | | | | | | | | | | 0.84 | | | | | | |
| | BROKEN | | | | | | | | | | | | 0.81 | | | | | | |

Figure C3. Tie Condition Index calculation form—improved accuracy.

5. The bad tie limitation factor (LF) can be calculated from either

$$\frac{63360}{\text{tie spacing in inches}} - 920, \text{ or ties per mile} - 920.$$

6. The preliminary index is calculated from the formula shown in Column 18.

7. The maintenance factor to allow for the number of clusters and the maintenance policy can be determined from Figure C4. Reading up from the proportion of bad ties to the appropriate maintenance cycle gives the MF on the y-axis.

8. The TCI is the preliminary index (Column 18) multiplied by the MF (Column 19).

## 4 DETERMINING THE BAD TIE LIMITATION FACTOR

The worst bad tie condition at which a track is considered usable is much less than 100 percent of the total ties in the track. Therefore, it is not possible to use the pure bad tie count to reflect actual tie condition.

The FRA's minimum safety standard for Class 1 track is five evenly spaced nondefective ties per 39 ft. Therefore, this would require 677 good ties per mile. To

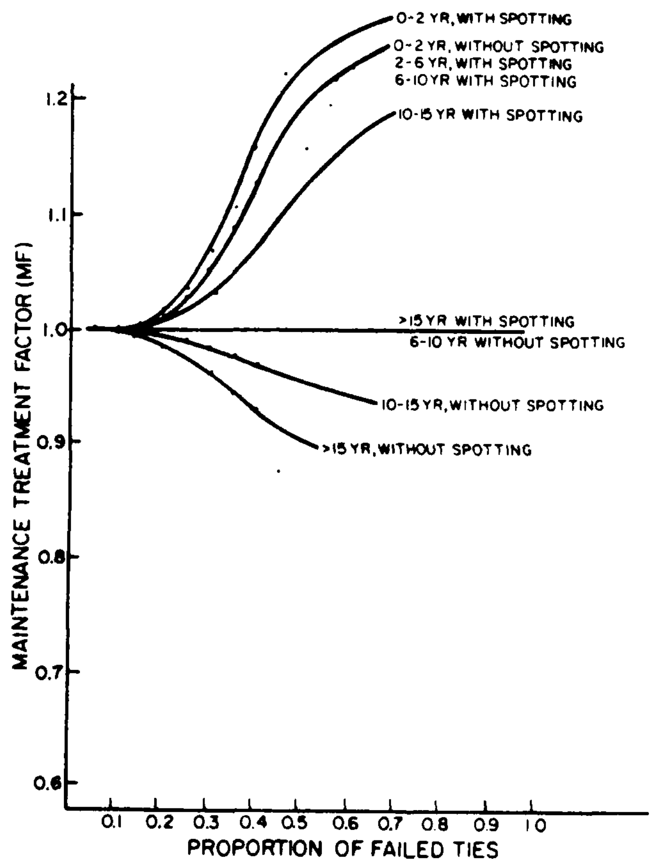



**Figure C4. Maintenance treatment factor vs. proportion of failed ties.**

ensure that there is a good tie for every joint, an extra 270 ties are needed for a total of 947 ties. However, since 27 of the evenly spaced ties would fall under a joint, the minimum good ties would be 920. Therefore, the maximum bad ties would logically be 2330 for 19.5-in. tie spacing. It is extremely unlikely that these ties would be exactly evenly spaced unless there was an extensive tie-spotting program. Actually, when adjusted, 2330 bad ties would give a TCI of about 18. This is a passable, but very unstable track, since with one more bad tie, the track would probably fail to meet FRA Class 1 safety standards.

Since the actual maximum number of bad ties depends on the total ties in a mile, the maximum must be adjusted. The maximum bad ties is, in effect, the Bad Tie Limiting Factor. To calculate the limiting factor, either of the following formulas should be used.

$$\frac{63360}{\text{tie spacing in inches}} - 920\text{, or ties per mile} - 920$$

For tie spacings other than 19.5 in., there is a slight error because there are varying possibilities that one of five good ties per rail length will fall under a joint. However, this error can be considered insignificant.

## 5 DETERMINING AN OPTIMAL SAMPLING PLAN FOR FAILED TIES

Any strategy for counting failed ties short of a 100 percent sample carries an expectation of error. For the standard railway practice of counting 100 ties at each milepost, this error is potentially very large, especially for relatively short stretches of track. A well-planned, statistically based scheme will both increase accuracy and minimize data collection effort.

The design of a sampling plan starts by specifying the permissible error on the accuracy and precision with which overall failed tie counts are to be estimated. The magnitude of the error of estimation will be affected by:

1. High inherent variability in the proportion of failed ties within a given mile.

2. The necessity to sample groups of consecutive ties as an alternative to random sampling of individual ties.

The sampling plan must also consider that the size of the tie population being sampled will vary greatly.

### Minimizing Systematic Error

To meet the given error specification, systematic errors must first be minimized. This means that the segment being sampled should be as homogeneous as possible with respect to subgrade, drainage, and track curvature. This specification dictates that separate tie counts be made for lengths of track sharper than 2 degrees and where changes in track foundation are apparent. As the probability of occurrence of changes in foundation characteristics increases with the length of the tie data segment, an upper limit of 3 miles should be used for segment length. Within this distance, nonconsecutive segments of tangent track or curvature could be combined to produce an aggregate failed tie count.

A randomized sampling technique in which perhaps every tenth tie is sampled would contribute greatly to minimizing systematic error in the inferred failed tie count. While this is the usual technique of sampling within other disciplines, it is simply not feasible from a logistics standpoint for tie inspection. The alternative is to count randomly selected groups of consecutive ties. With this technique, termed "group sampling," the inspector starts at a random location near the start of the segment. He/she then counts groups of, say, 50 ties at roughly equally spaced intervals. As long as there is variability between tie groupings, losses in reliability can be offset by increases in the number of ties sampled.

To illustrate the suitability of group sampling, calculations of variability were made for samples of 50 ties chosen at random from a mile of northeastern U.S. track, randomly selected from an available database of tie counts. The results are shown in Table C1. In this example, the within-group standard deviation exceeded the between-group standard deviation. This is a curious result if bad ties are randomly distributed, but a natural result of the tendency for bad ties to occur in clusters, since a bad tie transfers load to adjacent ties, thus hastening their deterioration. This tendency is reduced by deliberate spotting in of good ties, but will likely still be prominent if the size of the sampling group is large. As a result of high local variability, group sampling of ties may actually be more accurate than random sampling. Trials with live data have shown that tie-sampling groups made up of more than 50 ties produce the necessary within-group variability for this sampling method.

### Tailoring Sampling to Accuracy Specification

If failed tie sampling is based on counts of randomly distributed groups of 50 consecutive ties and is stratified into roughly homogeneous track sections, systematic error can be assumed to be negligible. Therefore, it can be assumed that failed tie proportions will be distributed according to the binomial distribution.

The sampling precision of a proportion of failed ties sampled in groups of consecutive ties depends on both the total number of ties counted and the number of

**Table C1**

**Analysis of proportion of bad ties variability for Sample of Five Groups of 50 Ties From Same Mile of Track***

| Group | Mean proportion of bad ties in group | Within group standard deviation (S.D.) of bad ties* |
|---|---|---|
| 1 | 0.26 | 0.09 |
| 2 | 0.28 | 0.08 |
| 3 | 0.24 | 0.17 |
| 4 | 0.16 | 0.13 |
| 5 | 0.36 | 0.11 |

*Excluding curves greater than 2 degrees.
Between group S.D. = 0.07
Within group S.D. = 0.12

groups. The total number of failed ties that must be counted can be calculated by the normal approximation to the binomial distribution as:

$$N = \frac{P(1-P)}{\frac{(SE)^2}{(t)} + \frac{P(1-P)}{N}} \qquad \text{[Eq C1]}$$

where:

P = the expected maximum proportion of bad ties

SE = the desired sample precision

t = a confidence level factor

N = the total number of ties in the track segment.

N would be considered the minimum total number of ties required to be counted to achieve a given percent accuracy, SE, at a given confidence level.

The procedure for calculating the number of groups required is similar. Here, N becomes the number of groups that must be sampled and P(1-P) is substituted by the actual variance between groups. The formula for the required number of groups, ng, is:

$$ng = \frac{t^2_{adj}}{SE_{\bar{x}}^2} \cdot S^2\left(1 - \frac{ng}{(Ng)}\right) + 1 \qquad \text{[Eq C2]}$$

where:

$t_{adj}$ = the t-statistic corresponding to the selected confidence level, adjusted for the small sample size

S = between-group standard deviation

n = number of sampling groups

N = total number of groups in track segment

= (Ng x cluster size)/(number of ties in segment)

$SE_{\bar{x}}$ = the specified permissible sampling error.

This leads to the paradoxical situation in which the between-group standard deviation must be known before the number of groups that must be counted can be calculated.

The accepted technique is to use Eq C1 to calculate the total number to be counted. This number is then divided by the number of ties in a cluster to estimate the total number of clusters to be counted. Properly, a post-audit should then be done on the expected precision of the estimate of the true proportion of failed ties by substituting the between-group standard deviation calculated for the sample. Practically, it is possible to use typical ranges of standard deviations from past experience to develop a general specification on the number of sampling groups regained.

### Calculation of Required Sample Sizes[24]

Once a permissible error is specified, the above formulas can be used to calculate required sample sizes. A reasonable error specification would be 90 percent confidence, with the sample producing an estimate of the population mean proportion of failed ties that is within $\pm$(0.2 x % failed ties) of the true proportion. For example, if the true proportion of failed ties were 40 percent, the inspector would be 90 percent confident that this would be estimated within $\pm$(0.2 x 40 percent) = 8 percent. Use of Eq C1 also requires an estimate of the maximum proportion of failed ties. The conservative assumption is made that the failed tie count would be less than or equal to 40 percent, or greater than or equal to 60 percent, yielding a maximum value of p(1-p) of 0.24. Finally, it is assumed that the average length of track segment is 1/2 mile. Eq C1 dictates that in the worst case, at least 300 ties must be counted in each track segment to achieve the above specifications for precision.

The estimate is not very sensitive to any quantity except the desired sampling precision, SE. The more restrictive specification is on the number of groups that must be sampled within a track segment. This calculation is best illustrated in Figure C5.

Referring to this figure, if the bad tie count were 25 percent failed ties and the standard deviation of the proportion of bad ties between sampling groups in this segment were 6 percent, six groups would be required. If the standard deviation were 10 percent, the sampling would require 10 groups. This result is roughly the same, regardless of the track segment's size. As the mean proportion of bad ties increases, a larger standard deviation is permitted for the same number of sample groups.

The ideal sampling method would have the inspector return to the site for more sampling after he/she had calculated the between-group standard deviation. However, in practice, some general guidelines can be established empirically. Trials with randomly selected segments from the AAR tie database have been performed and have led to a rule of thumb that between-group standard deviation is roughly 20 to 30 percent of the mean proportion of failed ties. Use of this rule allows development of a standard sampling plan using six groups. Because of the low sensitivity of the required number of groups to the segment length, this rule would stand, regardless of the track segment length, up to the 3-mile limit.

### Recommended Sampling Plan

It is recommended that sampling of bad tie counts be based on the following specifications:

1. Separate determinations should be made for tangents vs. curves greater than 2 degrees and for track exhibiting different drainage and/or subgrade characteristics. Since there are economies of scale on larger sample sizes, tangents interspersed with curves should be aggregated where possible.

2. Inspection should be in groups of 50 consecutive ties. These should be inspected by beginning the first group at a random location, near the start of the segment, interspersing the remaining groups at roughly equal intervals, corresponding roughly to

[24] H. Arkin, *Handbook of Sampling for Auditing and Accounting*, second ed. (McGraw-Hill, 1974), p 179.

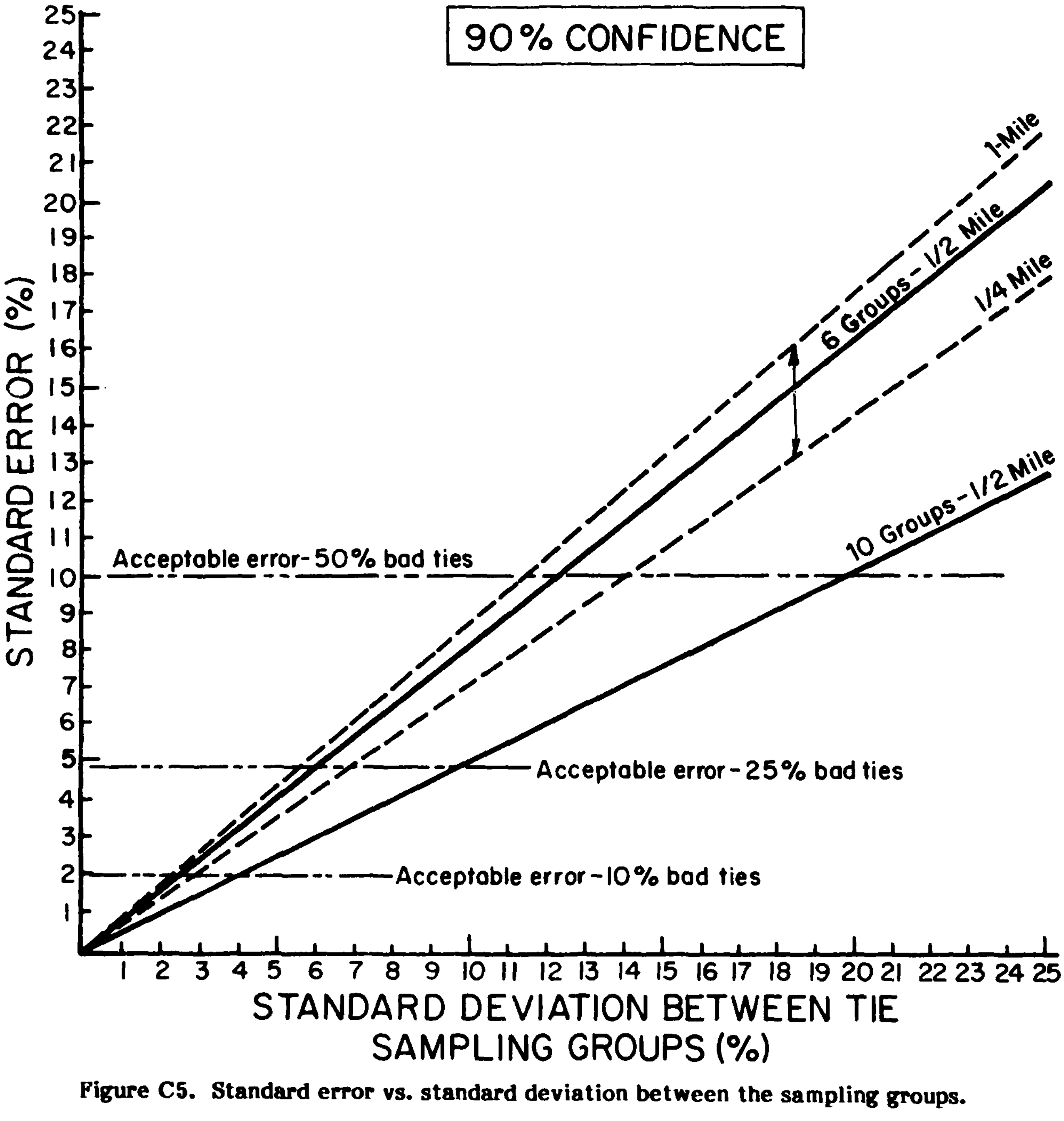



**Figure C5. Standard error vs. standard deviation between the sampling groups.**

0.8 x segment length/(number of clusters + 1). Sample spacing can be referenced to pole lines, but sampling should start at a random distance from the pole.

3. A minimum of six groups of 50 ties should be counted in any aggregate track

4. Extra groups should be counted if the inspector notices unusually large variation in the proportion of failed ties in different groups within the same track segment.

5. If the track segment length is less than 1/4 mile, the required groups can be reduced to four.

### Sensitivity of Sampling Plan

Figure C5 illustrates the sensitivity of the number of sampling groups to the specified sampling error (for a given between-group standard deviation). The number of sampling groups required is roughly inversely proportional to the permissible sampling precision; however, a 300-tie sample of six to 50 tie groups should be considered a minimum for 1/2-mile sections. The number of groups is very insensitive to segment length. If, say, 99 percent confidence were required on the $\pm 20$ percent standard error, the number of groups would have to be doubled (e.g., from six to 10 groups).

## 6 DERIVATION OF ADJUSTMENT FACTORS FOR THE BAD TIE COUNT

When using bad tie counts to develop a TCI, adjustment must be made to account for the fact that although a tie may be classified as defective, it will characteristically provide a certain level of support. The extent of this support, which varies according to the mode of failure, is important information when rating the priorities for tie programs between different rail lines. Adjusting bad tie counts for the support provided by defective ties gives a true indicator of how severe the loss in tie support capability is.

Ties must be able to distribute axle loads effectively to the ballast and maintain vertical stresses at the tie/ballast interface at acceptable levels. They must also provide a safety factor against high lateral forces, providing both a resistance to rail overturning and a resistance to lateral movement of the rail base. The extent to which a defective tie will provide these functions depends on the nature of its degradation.

### Load Support Provided by a Single Tie

To estimate the loss in support capability, an analysis was first performed to characterize the relative contribution of the components of a single tie to vertical and lateral rail support.

The assumptions of the analysis are:

Typical maximum wheel load = 27.5 kips
Rail type = 90-lb section
Track modulus = 2000 lb/in./in.
Tie spacing = 22 in.

All track deformations occur within elastic range, permitting use of elastic support theory.

### Vertical Plane

Under the above assumptions, and assuming uniform track conditions, a single tie in good condition will deflect an amount, $Y_o$, that can be calculated as:

$$Y_o = \frac{P \times C_I \times C_d}{\sqrt[4]{64EIU^3}} \qquad \text{[Eq C3]}$$

where:

$P$ = applied wheel load in pounds

$C_I$ = dynamic impact factor

$C_d$ = factor to add influence of adjacent wheel loads

$E$ = Young's Modulus = $30 \times 10^6$ lb/sq in.

$I$ = vertical rail moment of inertia in inches = 38.7 in.

$U$ = modulus of track deformation in pounds per square inch.

Under the above assumptions,

$$Y_o = \frac{27{,}500 \times 1.35 \times 1.22}{\sqrt[4]{64 \times 30 \times 10^6 \times 38.7 \times 1000^3}}$$
$$= 0.488 \text{ in.}$$

A track modulus of 2000 lb/sq in. with a tie spacing of 22 in. would correspond to a spring constant per individual tie, $K_{TOT}$, of

$$\begin{aligned} K_{TOT} &= U \times S \\ &= 1000 \text{ lb/sq in.} \times 22 \text{ in.} \\ &= 22 \text{ kips/in.} \end{aligned}$$

Therefore, an individual tie will provide a total resisting force of

$$0.488 \text{ in.} \times 22 \frac{\text{kips}}{\text{in.}} = 10.7.$$

The track spring constant of a 22 kips/in. tie is made up of the series equivalent of a tie stiffness of 580 kips/in. and a ballast/subgrade stiffness of 2.3 kips/in.

**Lateral Plane**

Rail can move laterally, either through rotation or translation (lateral movement of the base). The two movements combine to produce a dynamic wide gauge, which is an indicator of the tie's strength in the lateral plane. The amount of rotation depends on the pullout restraint of the gauge spike and the rotational stiffness of the rail. Translation is resisted by both gauge and field spikes and by friction between the plate and the tie.

Under most wheel passages, ties actually have little to do with restraining lateral loads. The critical factor is the ratio of lateral to vertical load (L/V) under a given wheel.

When the L/V is less than about 0.64, rotation is overcome by the restraining movement set up by the vertical wheel load. Beyond the L/V of 0.64, the resultant of vertical and lateral loads passes outside the rail base, and rotation can begin. However, when L/V exceeds 0.35 (the coefficient of friction between tie and tie plate), some lateral movement of the rail can occur.

In practice, L/V ratios are constrained by available friction between wheel and rail. Therefore, very large L/Vs of the type required to generate significant rail lateral movement typically only occur with wheel offloading.

The critical case that places a premium on tie condition is the situation where a large lateral force occurs with almost complete wheel offloading. This is, therefore, the case that must be analyzed to determine the lateral restraint capability of defective ties.

Figure C6 illustrates a laboratory test performed by the AAR. This test showed that under a lateral load sufficient to cause 1/2-in. dynamic wide gauge on track with new ties, 91 percent of the gauge widening was due to rail rotation, with 9 percent due to rail translation. Calculation of the restraining movement, caused by the 135-lb rail, leads to the estimate that 47 percent of the resistance to rotation was provided by the field side spike on the tie at the site of the load application. Therefore, 85 percent of the ties' resistance to lateral loading under this critical condition is due to pullout restraint of the field side spike, while 15 percent is due to the lateral resistance of field and gauge side spikes.

**Loss in Function Due to Decay**

A tie exhibiting the characteristics noted under the decay category in Table C2 could be characterized as having a 75 percent reduction in tie spring constant due to degradation of the heart of the tie, as well as 1/4-in. play due to crushing in the tie seat.

Therefore, the new track spring constant for the decayed tie would be:

$$K = \frac{0.75 \times 580 \times 23}{23 + 0.75 \times 580}$$
$$= 21 \text{ kips/in.}$$

The vertical force developed by the decayed tie, given that the first 1/4 in. of deflection is now picked up by adjacent ties, would be:

$$(0.488 - 0.25) \text{ in.} \times 21 \frac{\text{kips}}{\text{in.}} = 5.0 \text{ kips}$$

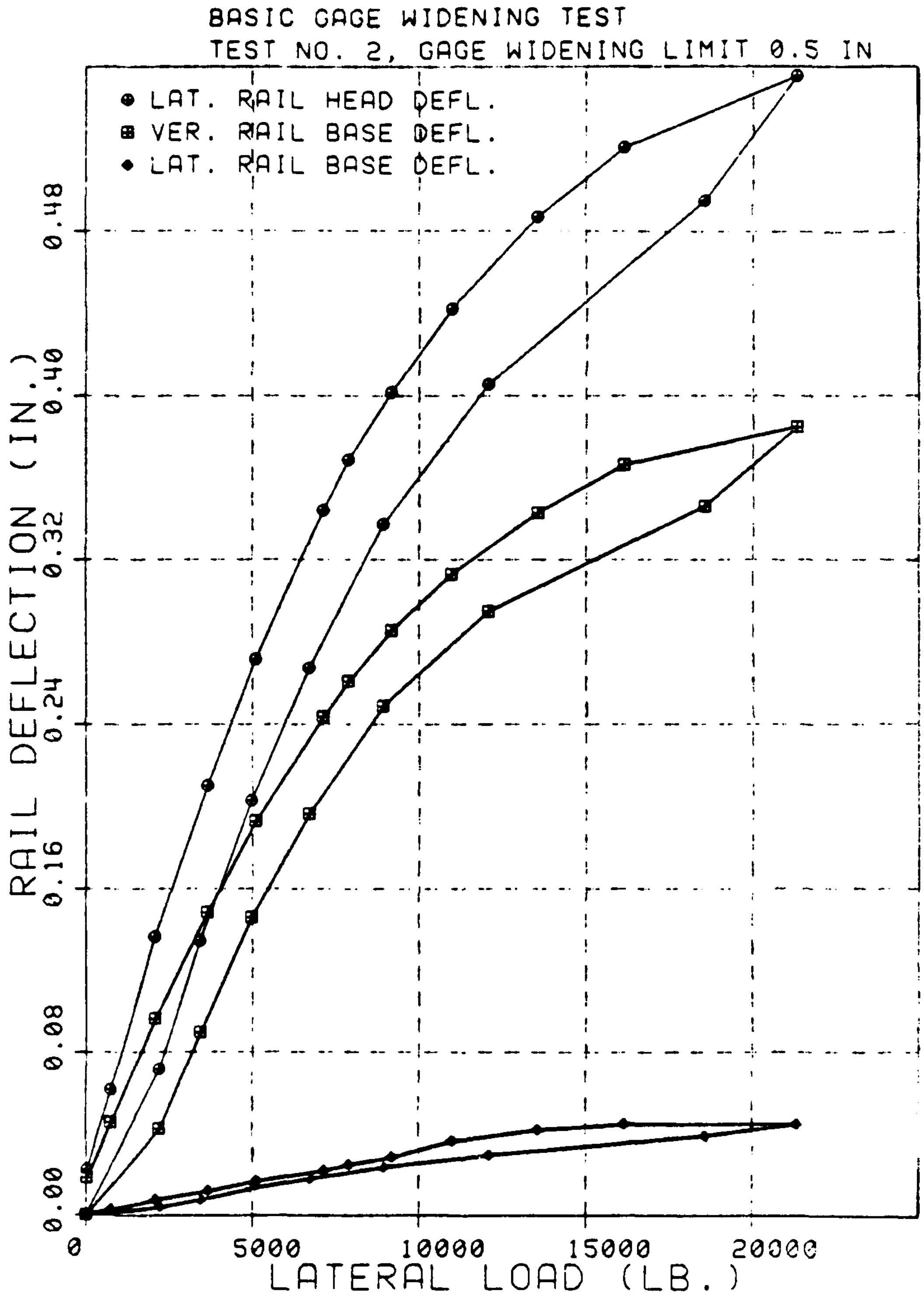



**Figure C6. Graph showing various rail deflections vs. latral loads, for zero vertical load.** (From A. Zarembski and J. Choros, *Laboratory Investigation of Track Gauge Widening*, AREA Bulletin 676 (January-February, 1980), Figure 3.

Table C2

Bad Tie Count Adjustment Factors

| Type of degradation | Description appearance* | % Loss in lateral restraint | % Loss in vertical support |
|---|---|---|---|
| Decay | - General destruction of wood fibers in plate area<br>- Plate crushing<br>- Hollowed center identifiable by sound<br>- Heavy checking<br>- Excessive plate movement<br>- Stringy wood fibers<br>- Separation of fibers in tie end | 79 | 53 |
| Splitting | - Longitudinal split on any of four sides of tie<br>- Ballast able to work through<br>- Split progressed through tie plate area<br>- Loss of holding power for one or more spikes | 46 | 38 |
| Spike killed | - Spikes showing signs of tilting to field side<br>- Spikes able to move<br>- Edge of tie plate shearing wood fibers<br>- General weakening of wood fibers in tie plate area | 84 | 0 |
| Broken | - Lateral split or crack adjacent to tie plate or near centerline of tie and progressing down through tie<br>- Drop of one end of tie relative to remainder of tie<br>- Hump in center of tie | 25 | 75 |

* Factors that may occur in various combinations. Any one is sufficient to signify a defective tie.

When compared to the 10.7-kip resistance offered by a good tie, this yields an estimated 53 percent loss in vertical support capability. A decayed tie would also typically show an 80 percent reduction in the pullout restraint of the gauge spike as well as an estimated 75 percent reduction in the lateral resistance of both field and gauge spikes. The result is a reduction in lateral support capability of 0.15 x 0.75 + 0.85 x 0.8 = 0.79 x 100 = 79 percent.

### Loss in Support for Split Ties

A split tie is less effective in spreading loads because its bearing capacity is proportional to the square of its width.[25] Therefore, a split progressing through the full effective bearing area of the tie's rail seat area, extending inward by an amount at least equal to (tie length - 4-ft 8-1/2 in.)/2 will result in tie support by two footings, each one half the width of the tie. This will halve bearing capacity. If it is assumed that 50 percent of the split ties are split through one tie end and the remainder through both tie ends, an average loss in vertical support capability of 38 percent can be derived.

To estimate the loss in lateral support capability for split ties, it is assumed that of the ties so marked:

25 percent have splits progressing through gauge spike

25 percent have splits progressing though field spike

25 percent have splits progressing through both spikes

25 percent have splits that do not affect either spike.

A split that has progressed through the gauge side spike hole is estimated to result in a 90 percent reduction in pullout restraint. A split through the field side spike will completely eliminate any effectiveness in lateral restraint capability.

Under these assumptions, ties identified as split will exhibit, on the average, a 46 percent reduction in lateral restraint capability.

### Loss in Support Capability for Spike-Killed Ties

Spike-killing has no effect on vertical support capability. However, a 50 percent loss in lateral restraint capability for both field and gauge spikes can be assumed, as well as an estimated 90 percent in pullout restraint for the gauge spike. This results in a typical loss in lateral support capability of 84 percent.

### Loss in Support Capability for Broken Ties

A flexural break under the rail seat--the typical mode of tie breakage--prevents any moment transfer to the end of the tie. Therefore, all rail seat support must be provided by half of the rail seat bearing area. This conservatively will double the

[25] G. Raymond and M. Roney, *Stress and Deformations in Railway Track*, Vol III (Canadian Institute of Guided Ground Transport, 1978).

pressure of the tie/ballast interface, and is equivalent to reducing the ballast/subgrade spring constraint under the failed tie of 50 percent. Therefore, the new spring constant is:

$$K = \frac{23/2 \times 570}{23/2 + 570} = 11.3 \text{ kips/in.}$$

If it is also assumed that a vertical play of 1/4 in. has developed due to tie settlement, the resisting force of the broken tie would be 0.238 in. x 11.3 kips/in. = 2.7 kips. This represents a 75 percent loss in vertical support capability.

In the lateral plane, tie breakage could be expected to result in the loss of 50 percent of the lateral holding capability of either the gauge or the field spike. The result is a 25 percent loss in lateral support capability.

## Application of Factors

Maintenance policy affects the occurrence of "clusters" of bad ties. Tie clustering is the result of the random statistical nature of tie failures and the transfer of extra loading from a bad tie to adjacent good ties. The accelerated degradation of surrounding ties produces a "cascading" effect.

A policy of renewing ties only when bad tie counts become high increases the number of tie clusters. Tie clustering can be minimized by frequently spotting in ties, which tends to break up clusters.

In the absence of information on the maintenance practices followed on a given rail line, a conservative approach must be taken when adjusting bad tie counts. It must be assumed that tie clustering does occur and that there is thus a high probability that a bad tie is adjacent to another bad tie. Under this assumption, a tie is considered good only if it provides both lateral and vertical restraint. This is logical since it cannot be assumed that adjacent ties will pick up deficiencies in support in either plane.

The percent losses derived earlier (Table C2) are a good proxy for the percent of ties classified as defective and not fulfilling their function. Therefore, under the conservative assumption, the proportion of defective ties in a given failure mode classification that are truly nonfunctioning, Pb, would be:

$$Pb = 1 - (1 - \%\text{ loss in lateral restraint})(1 - \%\text{ loss in vertical restraint}) \qquad \text{[Eq C4]}$$

Application of this equation produces the adjusted bad tie count factors given in Table C3.

Table C3

Adjusted Bad Tie Count Factors Under
Conservative Assumption of High Clustering

| Type of degradation | Bad tie count | Adjusted bad tie count |
|---|---|---|
| Decay | $N_d$ | $0.90\ N_d$ |
| Splitting | $N_s$ | $0.67\ N_s$ |
| Spike-Kill | $N_k$ | $0.84\ N_k$ |
| Broken | $N_b$ | $0.81\ N_b$ |

The total adjusted bad tie count is therefore:

$$0.90\ Nd + 0.67\ Ns + 0.84\ Nk + 0.81\ Nb$$

## 7 DETERMINING MAINTENANCE CYCLE FACTOR

An index of tie condition can be derived from bad tie counts according to the following formula:

$$\text{Tie Condition Index} = (1 - \frac{Na}{LF})\ (MF)\ (100\%) \qquad \text{[Eq C5]}$$

where:

LF is the bad tie limiting factor (the maximum number of bad ties theoretically possible, yet still providing a passable track).

Na is the adjusted bad tie count per mile and
$= 0.90\ N_d + 0.67\ N_s + 0.84\ N_k + 0.81\ N_b$

where:

$N_d$ is the number of decayed ties/mile
$N_s$ is the number of split ties/mile
$N_k$ is the number of spike-killed ties/mile
$N_b$ is the number of broken ties/mile

MF is a maintenance factor that considers the tendency of ties to cluster as a function of maintenance cycle.

### The Need for a Maintenance Cycle Factor

The rating given to ties in two different track sections having the same number of (adjusted) bad ties should differ if there is a significant difference in the percentage of failed ties clustered in groups of three or more. This is because clustering of failed ties results in local weaknesses in the track structure. The structural integrity of a track

structure depends on the rail's ability to transfer load from a poor tie to an adjacent good tie. Rail is very good at performing this transfer of wheel load. Depending on the size of rail, typically seven ties are significantly involved in supporting a wheel load.

Despite the ability to transfer load-supporting capability in the vertical and lateral planes, some loss in the ties' ability to perform their function occurs when clusters exceed two ties. This is manifested as higher local deflections in both planes, with a resultant higher rate of accumulation of permanent deformations. Soft spots develop which slightly increase dynamic loading. Although these clusters are tolerable, their existence represents a cost to the rail authority. This "diseconomy" should be reflected in the TCI which is intended to be used to allocate funds to produce maximum benefit.

The cause of significant differences in clustering is variation in both tie renewal policy and the number of years since the last tie renewal activity at a given site. This results from the natural tendency of clusters to grow and proliferate with the accumulation of load cycles. Load transfer causes incremental increases in the load that must be carried by adjacent ties, thereby increasing their rate of deterioration and transferring load to other good ties. The relative proportion of ties in clusters within a track segment is described by the statistic $\lambda$--the probability of finding a failed tie adjacent to another failed tie.

If it has been a long time since any ties were added, bad tie clusters will be more prevalent than if ties had been added recently. A policy of frequent spotting in of ties, which typically concentrates effort on breaking up tie clusters, results in a very low proportional occurrence of ties in clusters. Differentiation must therefore be made between these policies when rating maintenance priorities for track from a broad range of different locations.

## Calculation of a Maintenance Cycle Adjustment Factor

Two quantities must be calculated to arrive at an adjustment factor, MF. First, the relationship between maintenance cycles and the distribution of clusters must be estimated. Second, a weighting factor for the relative impact of different cluster sizes must be developed.

## Effect of Maintenance on Clustering

A survey conducted by the AAR Track Maintenance Planning Research Committee has shown that $\lambda$ will typically range from 0.5 times the proportion of failed ties, p, to 1.25 p, with occasionally large values of $\lambda$ when p is small. A clear stratification between replacement policies minimizing clustering and policies which leave large clustering was also noted.

Based on these observations, the schedule shown as Table C4 has been drawn up to represent the impact of maintenance policy on tie clustering. This table is weighted to more lightly used lines and will require upward adjustment of $\lambda$ if mainline traffic densities are being considered. Table C5 gives an example of the impact of $\lambda$ on the occurrence of clusters of different sizes, based on the assumption that failed ties are distributed according to the binomial distribution.*

*A review of results of the AAR survey shows this to be a valid assumption for 1.25 p.

## Weightings for Cluster Size

There is very little hard data on which to base a weighting of different sizes of tie clusters. Beam-on-elastic foundation theory states that a missing tie increases the loading of adjacent ties by 40 percent, and experience has shown that each additional tie in a tie cluster increases the incremental penalty in track deterioration. On this basis, the following is proposed as a weighting formula:

$$\text{WEIGHTING} = 1.5 \times (1.4)^{\text{no. in cluster} - 3} \qquad \text{[Eq C6]}$$

This weighting system is used by a Class 1 railroad to weight geometry defects detected by a track geometry car.

By combining this weighting with the factors presented in Table C3 and using the standard equation for the binomial distribution, it is possible to construct a reasonable

**Table C4**

**Estimated $\lambda$ for Different Times Since Last Tie Renewal**

| Years since last programmed tie renewal | With spotting of ties | Without spotting of ties |
|---|---|---|
| 0-2 | 0.25 | 0.5 |
| 2-6 | 0.5 | 0.75 |
| 6-10 | 0.5 | 1 |
| 10-15 | 0.75 | 1.1 |
| >15 | 1 | 1.25 |

**Table C5**

**Comparison of Occurrence of Cluster Sizes for Different Values of $\lambda$ for 50 Percent Failed Ties, 3200 Ties/Mile**

| $\lambda/p$ | No. of clusters of 3 | No. of clusters of 5 |
|---|---|---|
| 0.25 | 19 | 1 |
| 1 | 100 | 25 |
| 1.25 | 88 | 34 |

basis for adjusting the TCI for variation in maintenance treatment. Using the binomial distribution to develop cluster data for various proportions of bad ties and the above weighting formula for the effect of each cluster size, a computer program was written to develop Figure C4. For a given proportion of failed ties and the time since last tie installation (this assumed an average life of about 35 years), the effect of the maintenance policy--the maintenance factor (MF)--can be determined.

To determine the MF, it is necessary to find out the last time there was a significant replacement of ties (at least 90 percent of the bad ties) and when the last spot insertion occurred. In Figure C4, reading up from the proportion of bad ties to the appropriate maintenance cycle gives the MF on the y-axis.

## 8 DISCUSSION

The simplest index would be to use a percentage of bad ties in the track. In most cases, this would give an index that was higher than it probably should be. Example 5 in Table C6 shows a TCI of 34; however, if a pure percentage were used, the index would be 45.

The examples in Table C6 illustrate the TCI one might expect for typical track conditions. The difference between examples 1 and 2 is the type of defects, which, in this case, do not make a significant difference. However, as the tie condition deteriorates and more ties fail from decay, the difference in the index increases. Examples 4 and 5 show that the total number of ties per mile has a significant impact.

It should be noted that the TCI is an index of the present condition and does not consider the age of the good ties. This would be very hard to do without detailed data about when all ties were installed.

This TCI has been reviewed by the AAR Track Maintenance Committee and found to be logically correct. However, work is still needed to improve its logic and methodology.

Table C6

Effect of Some Variables Considered in Compiling Tie *Condition Index*

| Ex. no. | Type of track | Class | Bad ties | Percentage defect by type | | | | Years since last tie gang | Tie spot Yes/No | TCI |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Decay | Split | Spike | Broken | | | |
| 1 | Main line (3250 ties/m) | 4 | 600 | 5 | 25 | 60 | 10 | 4 | No | 79 |
| 2 | Branch line (3250 ties/m) | 3 | 600 | 50 | 20 | 10 | 20 | 6 | No | 78 |
| 3 | Light branch (3250 ties/m) | 2 | 1200 | 70 | 20 | 5 | 5 | 10 | No | 55 |
| 4 | Siding (3250 ties/m) | 1 | 1200 | 85 | 5 | 5 | 5 | 15 | No | 51 |
| 5 | Siding (2640 ties/m) | 1 | 1200 | 85 | 5 | 5 | 5 | 15 | No | 34 |
| 6 | Siding (2640 ties/m) | 1 | 1200 | 85 | 5 | 5 | 5 | 15 | Yes | 38 |

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