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TRACK STRUCTURES PERFORMANCE

COMPARATIVE ANALYSIS OF SPECIFIC SYSTEMS AND COMPONENT PERFORMANCE

FAST

FACILITY FOR ACCELERATED SERVICE TESTING

INTERIM REPORT
SEPTEMBER 1977

DEPARTMENT OF TRANSPORTATION ★ UNITED STATES OF AMERICA

Prepared by:

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16. Abstract

This report presents analyses of data, up to 50 million gross tons of traffic, from the Facility for Accelerated Service Testing (FAST). These analyses include concrete and wood tie track system performance and component performance, rail wear comparisons, ballast depth and type performance, ballast shoulder width, and subgrade performance. In addition, there is a summary of general FAST results to date which includes comments on the major maintenance performed on special trackwork and other sections of the track not specifically addressed in the main chapters of the report. Also, some of the information contained in this report was derived from data from other sources such as the Velim test track in Czechoslovakia.

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This report is the result of an extensive 4-month effort by a special Department of Transportation (DOT) team composed of individuals from the Federal Railroad Administration (FRA) Office of Rail Safety Research (ORSR), and the Transportation Systems Center (TSC). The team members were associated with the DOT/FRA Improved Track Structures Research Program.

The team cochairmen were Dr. R. Michael McCafferty of FRA and Thomas Comparato of TSC. They provided overall management direction. The principal chapters on the comparative analysis were written by D. McConnell, Dr. A. Kish, and A. Sluz of TSC, and by H. Moody of FRA. Dr. McCafferty contributed extensively to the final report composition through technical editing.

Data collection and reduction were performed by the DOT Transportation Test Center (TTC) personnel who where assigned to the Facility for Accelerated Service Testing (FAST) Program. In particular, the work of R. Gill of FRA and of R. Reiff, M. Levine, J. Venturella, D. Knight, G. Jorgensen, M. McClelland, and E. Bates of the TTC FAST operations and maintenance contractor, Dynalectron, was vital in providing processed data to the team.

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CHAPTER 1. INTRODUCTION

Late in 1976, the Federal Railroad Administration (FRA) of the Department of Transportation (DOT) initiated operation of the Facility for Accelerated Service Testing (FAST) at the Transportation Test Center (TTC) in Pueblo, Colorado. This facility was conceived as a joint venture of the FRA and the railroad industry and is briefly described in chapter 3. It was designed to rapidly assess the performance of railroad systems and components under greatly accelerated rates of service.

The FAST track incorporates a series of test sections intended to evaluate the influence of various factors on track performance. Some of the test variables included are: rail metallurgy, type of tie, ballast type and depth, and rail fastenings on wood ties.

Systems performance and component interactions are of particular interest to the participating members of the railroad industry as crucial issues in various aspects of track performance.

Also the Office of Northeast Corridor Project (ONECP) of the FRA has a definite interest in these issues. ONECP is charged with developing the engineering approach for upgrading the track in the corridor from Washington to Boston. Of particular interest were the comparative evaluation of the performance of wood and concrete tie track systems and the effectiveness of varying ballast type, depth, and shoulder width in improving the long-term behavior of track.

This report presents analyses of FAST data and includes, where necessary, other test data for the comparative evaluation of the performance of different track structures and components. These analyses were conducted by a special project team supporting the FRA Improved Track Structures Research Program consisting of staff from the Transportation Systems Center (TSC) and the FRA Office of Research and Development (OR&D). Initial processing of the data collected at FAST was done at the DOT Transportation Test Center.

The assessment of the comparative performance of the FAST track test sections was conducted on two levels. The first analysis level, which is discussed in chapter 4, is an evaluation of the overall or gross performance of the track systems under repeated loads of the rapidly accumulating traffic at FAST. In order to perform this analysis in quantitative terms, indices of systems level performance, closely related to measures of the adequacy of track in service, were defined. These indices represent objective measures of the capability of the track to provide stable support and guidance to trains including effects of track settlement, geometry deterioration, and track substructure performance. The principal analysis discussed in chapter 4 is the comparison of wood and concrete tie track.

Chapter 5 assesses track performance at the component level. In particular, it is concerned with the evaluation of concrete ties and fastener performance against specified indices of performance. Crosstie performance is evaluated in terms of flexural strength under load, tie deterioration and failure due to cracking, and excessive tie movement in service. Similarly, concrete tie fastener system performance is evaluated in terms of fastener and pad movement or failure, and the restraint provided to the rails against gage spread, longitudinal movement, and creep. The experience at FAST with the application of elastic fasteners to wood crossties is briefly reviewed with primary emphasis on the tie plate cutting behavior.

Chapter 6 presents the preliminary analysis of rail wear at FAST, specifically comparing the wear rates of various rail metallurgies and heat treatments.

In addition, the influence of the tie plate cant on rail wear is analyzed. The primary index of performance applied in this preliminary review was gage point wear with accumulated traffic.

The analyses of the performance of the components of the track substructure, ballast, subballast and subgrade, and their influence on overall track performance are discussed in chapter 7. The principal concern of these analyses is the nature of the characteristics of the substructure components and their behavior under load. In particular, the variations in substructure performance due to the influence of tie type, and the effects of ballast depth, type, and shoulder width are considered. This influence is analyzed against criteria based on pressures and permanent strains in the ballast and subgrade. In addition, evidence of the deterioration of the substructure materials was collected at FAST, and visual observations were recorded. This last activity was conducted primarily at locations where indications of substructure problems were evidenced, with an attempt made towards defining the degradation mechanisms.

The comparisons of track systems performance in this report are primarily based on the assessment of available data from FAST through the first 50 million gross tons (MGT) of testing. It represents the first detailed analysis of FAST results and their application to resolving the questions of track performance and upgrading. The results of these analyses are, in many respects, preliminary because data, in general, were limited to only 20 to 50 MGT. This represents only a small segment in the life of track, and only some 10 percent of the planned Phase I test duration at FAST. The trends indicated by this FAST data are preliminary, and as more information becomes available, future analyses must reevaluate and verify these trends.

In order to interpret these FAST trends in the context of revenue service and to project these trends into a longer term, additional sources of data external to FAST were integrated into the analyses. The primary sources for this data were the ongoing research activities of the Improved Track Structures Research Program and the published reports of the Office of Research and Experimentation of the International Union of Railways.

The results compiled in this report formed the basis for an oral report given to the engineering and management staffs of the Office of Northeast Corridor Project on April 8, 1977. This report comprises a detailed analysis of the data on which that presentation was constructed. As such, it represents a source of data for consideration by ONECP in planning for corridor upgrading and by the railroad industry in planning track maintenance and construction.

CHAPTER 2. EXECUTIVE SUMMARY

This report is a preliminary evaluation of the performance of specific track system and components on the Facility for Accelerated Service Testing (FAST) including support, where necessary, through observations and data from other track system studies and test tracks.

Major emphasis was placed on evaluating the system and component performance of concrete tie track including comparisons with similarly constructed wood tie track. Other areas of analysis were: performance of various ballast types and depths, the effect of ballast shoulder width on wood tie track, and wear rates of different rail metallurgies and heat treatments. Broad areas pertinent to the operation and maintenance of FAST are presented.

Chapter 8, Summary of Findings, summarizes the findings, trends, and conclusions up to the first 50 million gross tons (MGT) of traffic on FAST. In most instances, this time period represents only about one-tenth of the expected component life and only preliminary trends can be identified. It was not possible, in some cases, to define statistically significant conclusions.

The system analyses of concrete and wood tie track performance differences indicate:

a. More initial settlement of concrete tie track with new construction, however concrete tie track constructed on an old roadbed settles much less than concrete tie track on a newly constructed road bed.

b. Better uniformity of settlement of concrete tie track (improved track surface and profile). (Note ballast and subgrade comment page 2-2).

c. Greater vertical and horizontal track stiffness (less deflection with the same load) for concrete tie track. This should provide a track with greater vertical and lateral stability.

Concrete tie and fastener component performance of FAST was considered to be good. Comparisons with revenue service results indicate similar trends. Specific results and conclusions include:

a. Measured tie bending moments lower than American Railway Engineering Association (AREA) Specification requirements.

b. Fastener clip movement has not been a critical performance or maintenance problem.

c. Insulator breakage and movement has not been a critical maintenance or performance issue.

d. Spring clip type fasteners on wood ties appear to reduce tie plate cutting.

e. Indications are that large rail corrugations are detrimental to concrete tie track performance.

Analyses of the performance of subgrade, ballast types and ballast depths indicate the following:

a. Differences in settlement for various ballast types and depths (greater than 15 inches) are not significant.

b. There are no indications of track performance differences as a result of different ballast types.

c. Ballast degradation has occurred under concrete ties at "soft" spots which probably were the result of high impact loads. (e.g. corrugations, rail joints).

d. If the trend of high permanent ballast and subgrade strain (settlement) continues in the concrete tie section, it may become a source of concern.

The rail wear data indicates trends as follows:

a. The five types of rail can be divided into three groups for ranking of gage point wear; i.e., first head hardened and chrome molydenum; fully heat-treated and high silicon; and lastly standard rail.

b. Wear with 1:14 cant tie plates is greater than wear on 1:30 and 1:40 cant plates.

c. Rail wear on concrete tie track is slightly less than wear on wood tie track.

CHAPTER 3. FAST BACKGROUND AND DESCRIPTION

The Facility for Accelerated Servicing Testing (FAST) is a government-operated facility designed to compress time, as represented by railroad component and system life, by a factor of about 10 over that experienced in revenue service. The facility was recently constructed at the Transportation Test Center of the U.S. Department of Transportation near Pueblo, Colorado to provide a means of responding to the need for solutions to existing railroad problems. The project was conceived as a joint government/industry effort to respond to this need in track structures and rolling stock.

The initial group of meetings, among the Association of American Railroads (AAR), FRA, and industry representatives, was held in the fall of 1975. The intention of these meetings was to define FAST and its objectives. The track layout and definition of experiments were established based on not only technical requirements, but also on the available resources. For example, the closed loop was formed by adding new track to an existing track which was modified to become a part of the test facility. This greatly facilitated the construction of the track and defined the track layout around which the track experiments were to be designed. Likewise, the available materials for track construction were compiled and experiments planned based on research needs, the available space in the track, and resources. This was done for almost every aspect of the present FAST track experiments.

These early planning activities and the ongoing FAST program were under the auspices of the Track Train Dynamics (TTD) Phase II Program. TTD is an international government/industry research program sponsored by the Federal Railroad Administration, the Association of American Railroads, the Railway Progress Institute (RPI), and the Transportation Development Agency of Canada. In general, the FRA operates and maintains the facility while the industry provides the appropriate guidance, rolling stock, and where necessary, other components and materials.

The FAST organization, in a concise form, is shown in figure 3-1. Various other groups, committees, and individuals which are not shown in the figure support the FAST Program. The customers are emphasized in this chart to underline their importance to the implementation of the results from FAST. These results will aid and expand everyone's understanding of system and component performance and can contribute substantially to the improvement of the nation's railroad system.

FAST is a 4.8-mile loop (see figure 3-2) of which 1.8 miles are new construction, and 3 miles are on an existing roadbed that had 0.5 MGT of traffic prior to the time FAST was built. Over half the track (13,023 feet) lies on curves and spirals, and nearly 60 percent of the track has continuously welded rail (CWR). The grade and superelevation of the FAST track are as shown in figure 3-3.

As indicated in figures 3-2 and 3-3, the track is divided into 22 test sections, each of which is either a test of special track structure components or a transition zone between different test sections. These transition zones generally consist of spirals. All of the test sections are subjected to the same train operations, providing a basis for assessing track performance from section to section.

Documentation of the various test sections from which data were analyzed for this report is in appendix A, along with a list of pertinent measurements collected in these test sections.

FAST test operations began on September 22, 1976, less than 1 year after the track layout was defined. FAST consists of many different track and rolling stock experiments compressed into a short loop. Consequently its design and

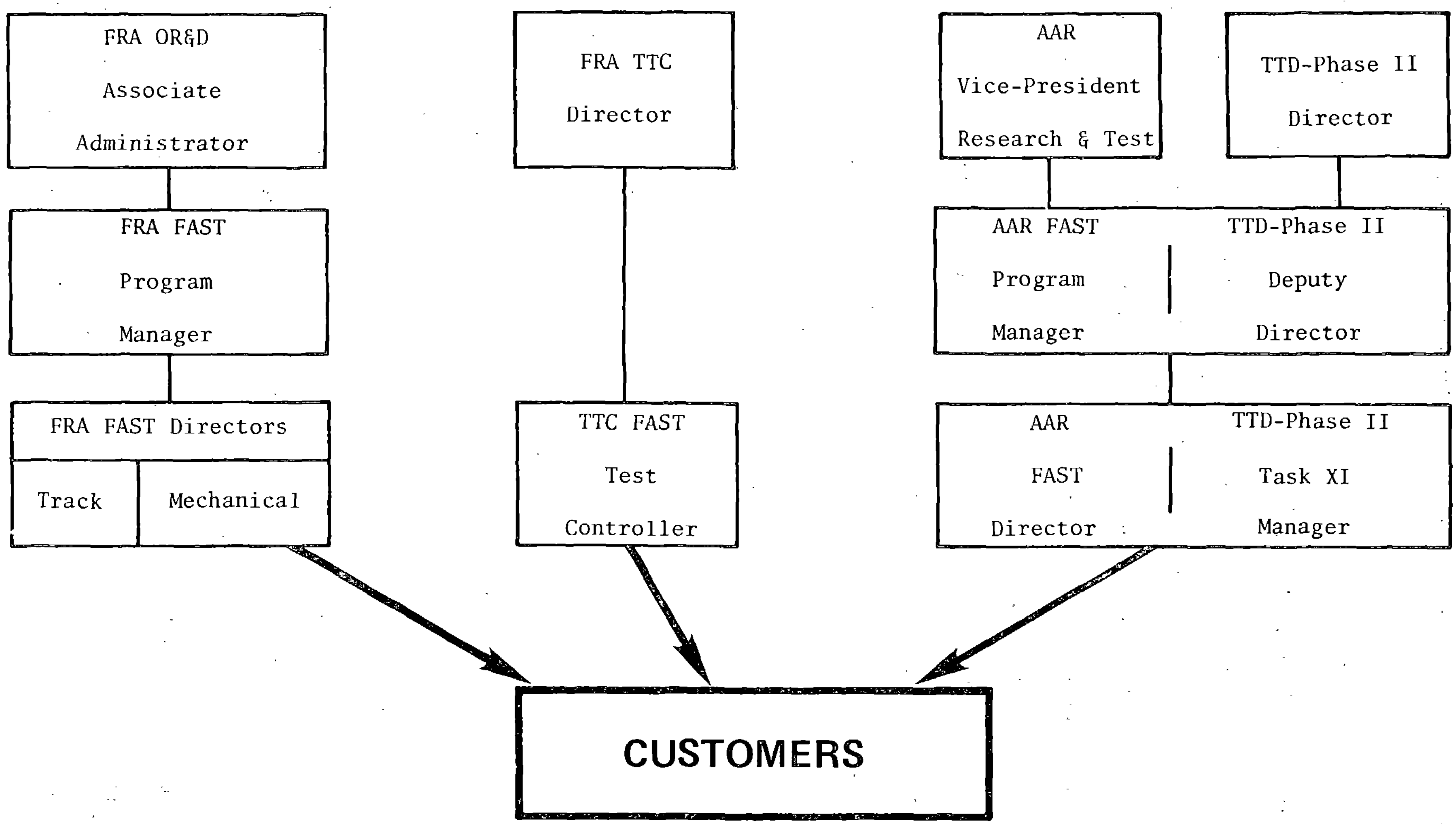
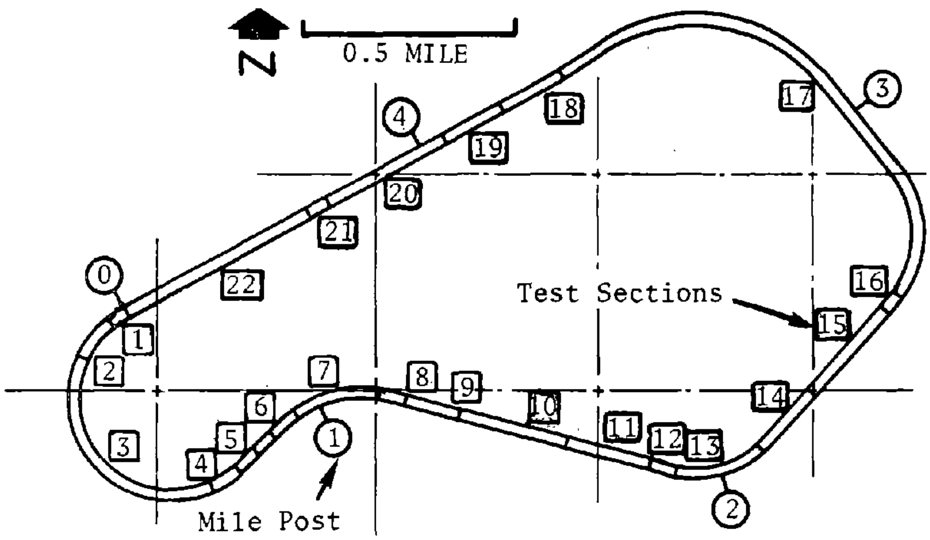


Figure 3-1. FAST Organization



Legend:

Test Section	Description/Test Variable
1	Existing No. 20 Turnout
2	Rubber Pads/Wood Ties
3	Rail Metallurgy, Tie Plate Cant, Ballast Shoulder Width, Spiking Pattern
4	Spiral, Standard Track
5	Bonded Joints
6	Steel Ties (removed at 28 MGT)
7	Fasteners/Wood Ties
8	Spiral, Standard Track
9	Reconstituted Ties
10	Elastic Spikes, Spring Frogs
11	Joints, Frogs and Guard Rails
12	Spiral, Standard Track
13	Rail Metallurgy, Spike Hole Filler
14	Existing No. 20 Turnout
15	Ballast Shoulder Width
16	Glued No. 20 Turnout
17	Concrete Tie Track
18	Ballast Depth
19	Oak and Fir Ties
20	Ballast Type and Depth, Rail Anchors
21	Welded No. 20 Turnout
22	Spiking Patterns, Rail Anchors

Figure 3-2. The FAST Track

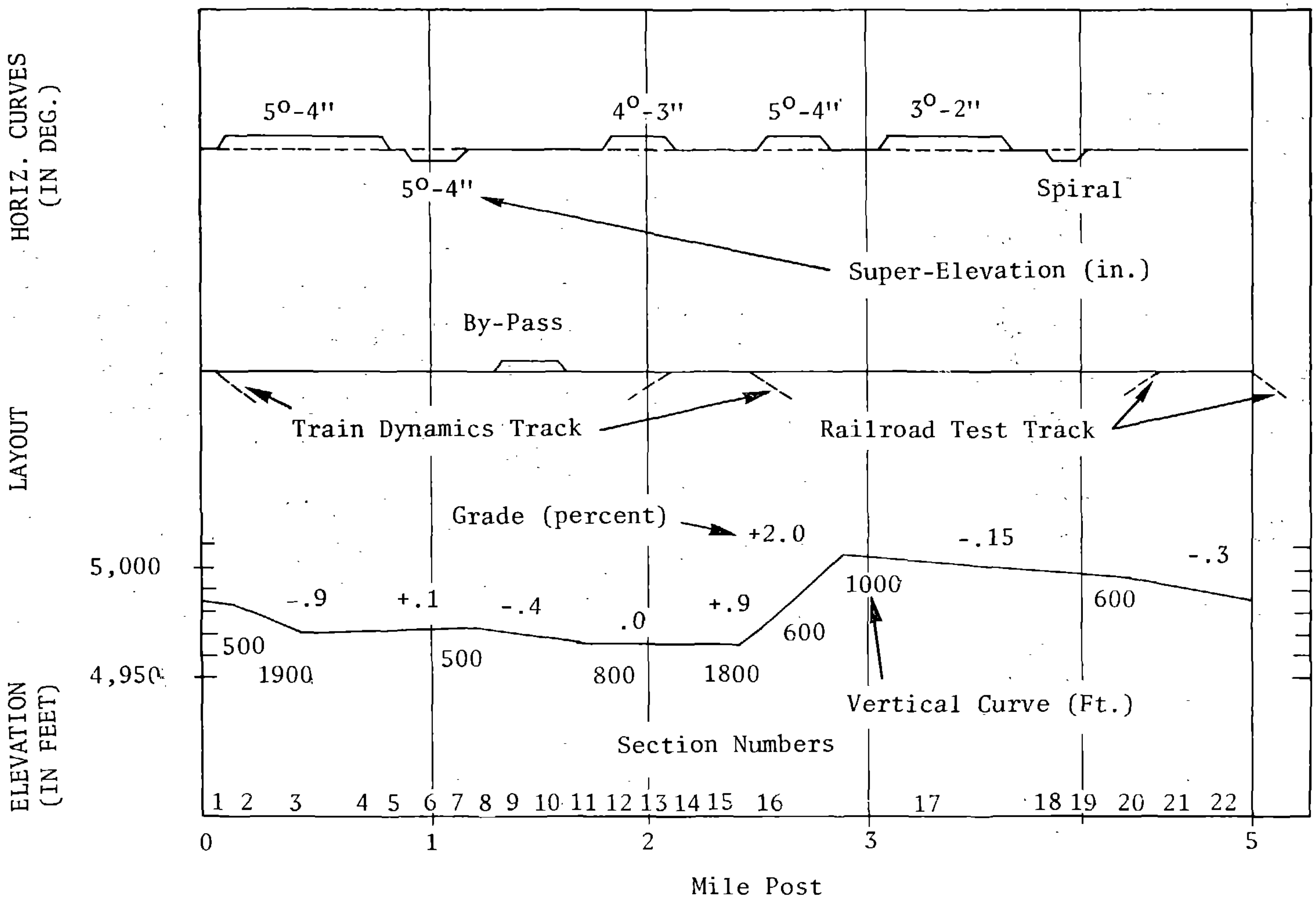


Figure 3-3. FAST Track Chart

operation are unique, and as such, care should be taken when comparing FAST to any other train operation in the United States.

This is obvious when one considers that the consist is a unit train composed of mostly 100-ton hopper cars and a few tank cars and other lighter cars. However, it is not the usual unit train that runs loaded in one direction and empty in the other. This consist is always loaded the same and is programed to operate around the FAST loop for about 100 times each night. The following night this train runs in the opposite direction for an equivalent number of laps. While the cars are rotated within the consist, this type of operation does create special circumstances which need to be considered when analyzing track and rolling stock results. The FAST consist started with a majority of the cars having new wheels or with wheels that had been turned to a new profile. This created a consist with most of the cars having a constant flange height. This condition, coupled with the fact that the original rail had little or no wear, resulted in a circumstance which requires qualifying the results of the rail and wheel data analyses.

Slightly less than one-half (about 44 percent) of the loop consists of tangent track which is far less than the normal operating railroad situation. Also, FAST operations are limited to speeds in the 40 to 50 mph range. The curves were designed based on a 2-inch unbalance. With a two percent grade and several diverging moves through number 20 turnouts, the average speed of operation depends upon the direction of travel and is between 42 and 44 mph. This limits the test variables and may add to the complexity of comparisons with revenue service.

Pueblo, Colorado is a semi-arid environment and the FAST track location has a particular soil condition reflecting that region. This combination provides a very stable subgrade support for the FAST track. It should be pointed out that the FAST embankment conditions are a variable. Some of the roadbed was built in 1973 and has been subjected to several winters and long-term settlement. This is opposed to sections 17 through 20, which at the time of initial operations, had not experienced a complete, yearly weathering cycle.

Combining these effects and other component differences among the many test sections and the overlap of experiments, it is evident that judgment and caution must be continually exercised when drawing conclusions from the test data. It would be relatively easy to compare two components or systems tested at the same location, but this has not been the case after only 50 million gross tons. In future tests, it may be necessary to have the tests rotate locations in order to truly compare one variable with another. Likewise, a conversion or translator factor to revenue service requires complete knowledge of both situations. This relationship could develop as FAST results are compared to or supported by studies of the TTD and Improved Track Structures Research Programs.

CHAPTER 4. WOOD AND CONCRETE TIE TRACK SYSTEM PERFORMANCE

The overall assessment of the performance of wood and concrete tie track systems under equivalent service conditions can be performed on two levels. The first level is an evaluation of the gross performance of the track system under the repeated loads imposed by accumulated traffic. The second level is an evaluation of the behavior of individual track structural components, such as rails, ties, fasteners, and ballast, and assessing the ability of these components to survive in the track load environment without significant structural distress. The comparison of the gross or overall system performance of concrete tie track construction is the primary concern of this section. The component level analyses of the second type are presented in chapters which follow.

4.1 Approach to Track Systems Evaluation

In order to evaluate and compare the performance of these two track structural systems in quantitative terms, indices of system level performance should be closely related to measures of the adequacy of track normally applied in revenue service.

Described in terms of its functions, track can be defined as a structure to supply stable support and guidance to trains at minimum cost. The principal characteristics of track system performance under this functional definition, support, guidance, and economy, can be interpreted in terms of readily definable track parameters. The stability of the support provided to wheel loads by the track structure can be determined by the evaluation of the ability of the track to retain track geometry, especially profile and surface. Directly related to these parameters are the rate and uniformity of track settlement under traffic. These two sets of measures differ primarily in the frame of reference employed in evaluating vertical track irregularities. As will be shown, vertical track resiliency can be directly related to long-term track deterioration.

Consequently, vertical track resiliency represents not only a measure of track condition, but also a basis for projecting long-term performance.

The guidance function of the track system is most directly related to the behavior of the track in the lateral or horizontal plane. In this instance, the retention of alinement, gage, and crosslevel are the primary characteristics of overall track performance. Again the resiliency of the track, in this case horizontal resilience, represents a measure of the capability of the track to restrain the lateral guidance forces generated at the wheel/rail interface without deterioration of the track geometry.

Finally, a tangible characteristic of the elements of track costs which are influenced by the type of construction is embodied in the maintainance activity which the track system requires to meet the demands of a given level of train service. For two equivalent track constructions employing wood and concrete ties and subjected to identical service environments, both the frequency and scope of maintenance activity required are characteristics of track system performance. These measures can be directly related to the total costs of providing support and guidance to trains.

Specific indices of overall track system performance may be identified for each of the measures identified above. These indices, which are discussed in subsequent portions of this section, were adopted as measures of the condition of a track section at a point in time and as indicators of the rate of deterioration of the track sections under study. These indices, which are utilized in comparing wood and concrete tie track system performance in the following sections, are summarized in table 4-1 along with the sources of the data which were evaluated.

As this table indicates, the comparison of the overall system performance of wood and concrete tie track construction is based primarily on the

Table 4-1. Track System Performance Indices

TRACK PERFORMANCE CHARACTERISTIC	PERFORMANCE INDEX	DATA SOURCES UTILIZED
Geometry Deterioration with Traffic	Track Shift Index Track Roughness Index	FRA FAST Track Velim Test Loop ORE Mainline Track Florida East Coast Railway Company
Rate of Track Settlement		FRA FAST Track Velim Test Track
--Overall	Mean Vertical Settlement Relative to Fixed Benchmark	
--Differential	Standard Deviation of Vertical Settlement	
Track Resiliency		FRA FAST Track C&O/B&O Tests
--Vertical	Vertical Track Modulus	
--Horizontal	Horizontal Track Modulus Horizontal Load at Yield	
Economic Performance	Maintenance Requirements and Cycles	Operating Railroads

assessment of available data from the FAST track in Pueblo, Colorado. These data were evaluated to define, as far as practicable, the initial trends in track performance which the test track section exhibited. However, the data available from FAST covered only the first 45 MGT of testing. This period represents only the first 10 percent of the total duration of the planned test, and as such, is strongly influenced by initial construction conditions.

In order to interpret these initial trends, additional sources of data external to FAST were employed. The primary sources for these data were the ongoing research activities of the FRA Improved Track Structures Research Program, and the published reports of the Office of Research and Experimentation of the International Union of Railways. This general approach to evaluating FAST data against other data sources was employed as a means of interpreting FAST loop test data in the context of normal revenue service. This translation is especially crucial since loop tests on track generally offer conservative or pessimistic predictions of main line track performance.[1]

In order to isolate, to the extent possible, the influence of tie type on FAST track performance, two principal track test sections at FAST were evaluated. These track sections are designated as sections 3 and 17 in the FAST track description in appendix A. While differing in the specific details of construction, these sections represent the most nearly comparable sections within FAST in which tie type is the primary variable.

Sections 3 and 17 were both constructed for similar types of train service, but as shown in Table 4-2, there is a difference in grade which will affect train speed and dynamic loading conditions. Although section 3 was constructed primarily as a test of rail metallurgy, evaluation of the characteristics of sections 3 and 17 generally supports the use of these two sections in the

Table 4-2. FAST Test Section Properties
Concrete vs. Wood Tie Construction

FAST TEST SECTION	TIE TYPE	BALLAST TYPE	BALLAST DEPTH	SHOULDER WIDTH	TIE SPACING	FASTENER	RAIL SIZE	RAIL JOINTS	CURVATURE	GRADE
3	Wood	Slag	15"	Varies 6"-18"	19.5"	Cut Spike	136RE & 132RE	CWR	5°	-0.9% TO +0.1%
17-Curve	Concrete	Granite	15"	12"	24"	Elastic	136RE	CWR	Spiral & 5°	+2.0%
17-Tangent	Concrete	Granite	15"	12"	24"	Elastic	136RE	CWR	Tangent	+2% TO -0.2%

the evaluation of wood and concrete tie track performance. However, it is significant to note that section 3 was constructed by replacing the rail and tamping of an existing track.

Consequently, while section 17 was constructed on a newly graded embankment, the embankment under section 3 was part of the original Train Dynamics Track, and as such, experienced 3 full years of weathering and cementation as well as approximately 0.5 MGT of traffic. This factor is significant in comparing the settlement behavior of the two test sections, especially in light of the nature of the cementing sand subgrade which exists at FAST.

The sections which follow present the results of the analysis of these two test sections against each of the criteria presented above. In each case, FAST data were employed in the initial review, with other data being incorporated to interpret and clarify the results as indicated earlier. The single exception to this practice is the review of FAST maintenance data.

Because FAST was subjected to only 45 MGT of traffic up to the initiation of this report, there has been virtually no scheduled out-of-face maintenance performed in any of the test sections under consideration. Consequently, there is very little that can be noted on maintenance cycles for concrete versus wood tie track systems.

The exception to this situation is the maintenance required by problem spots in the concrete tie test section. In the area of these soft spots, substantial ballast deterioration and tie skewing have been noted. These problems, which are analyzed in detail in the latter portions of this report, are considered as local problems and do not, as yet, appear to be indicative of overall systems performance.

4.2 Analysis of Track Settlement

Track settlement data provides a measure of the stability of track under repeated train loads. Settlement is defined as the change with traffic of the elevation of the track from the as-built position, as measured from a fixed benchmark located adjacent to the track. Consequently, settlement trends provide an overall measure of the effectiveness of a track configuration in distributing wheel/rail loads to the roadbed without overstressing of the track structure foundation.

A basic index of the settlement behavior of a track section can be defined as the mean elevation change for a series of survey points within the track section of interest. Figure 4-1 compares the mean incremental settlement for two of the 5° curves within FAST of wood and concrete tie construction. The trend which is emerging from this data indicates greater overall settlement of the concrete tie track as compared to the wood tie track. This shift of the two curves is largely a result of the greater settlement of the concrete tie track during the first 20 MGT of traffic at FAST with virtually identical average rates of track settlement with accumulated tonnage for both sections in the period from 20 to 40 MGT.

This behavior may be due in part to the new embankment under the concrete tie track, which was described earlier, as opposed to the weathered roadbed below most of the wood tie track. This settlement data is not suprising in light of this difference.

Similar trends through 33 MGT are indicated in figure 4-2 which presents mean settlement data for the ORE Track Test Loop located at Velim, Czechoslovakia. Again concrete tie track shows a greater overall settlement with a comparable rate of settlement with the wood tie track after an initial period. It should be noted that operating conditions at Velim and FAST are similar, with both facilities seeing an average of 1 MGT of traffic per day operating over roughly

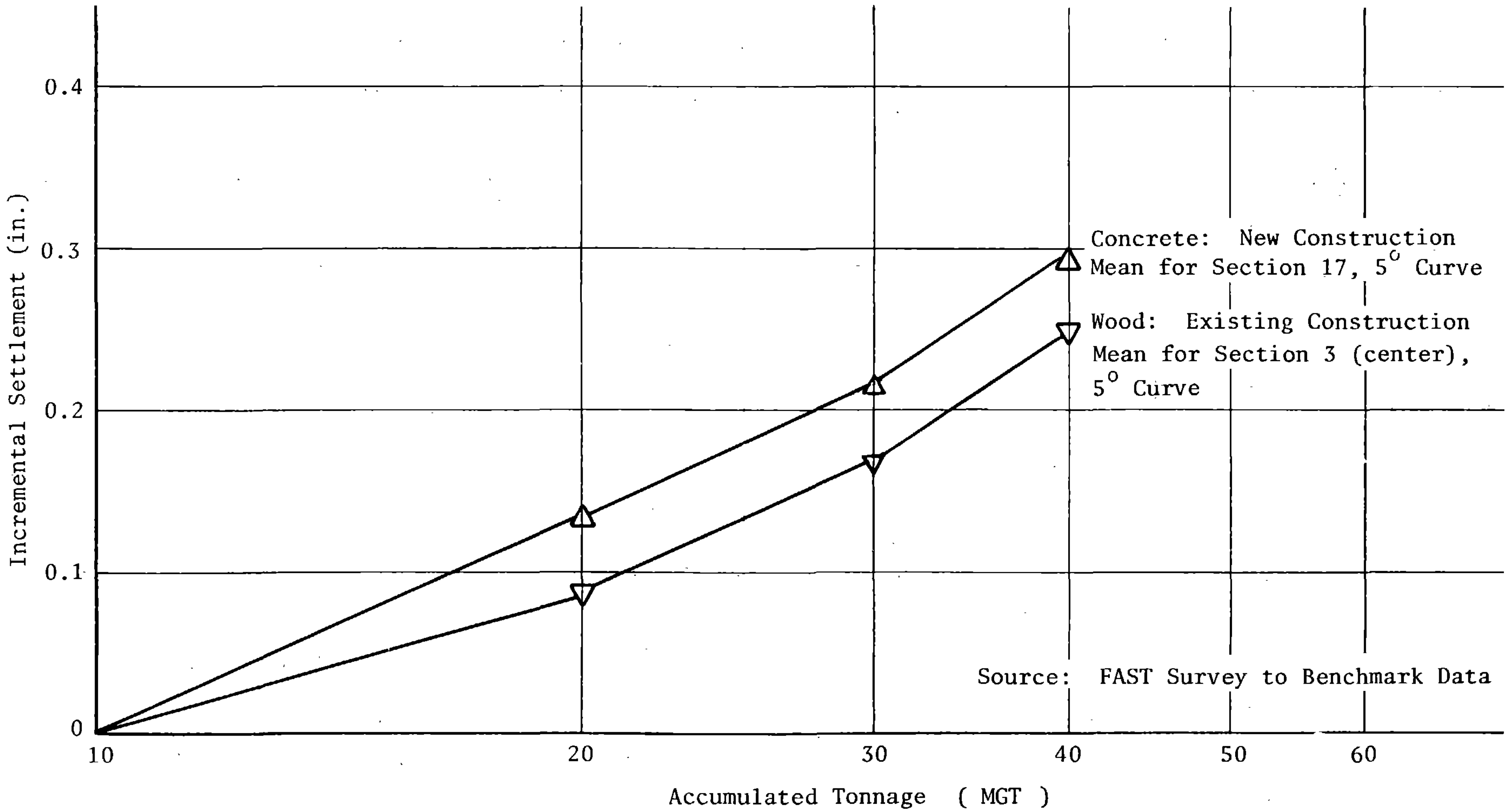


Figure 4-1. Incremental Track Settlement for FAST Track

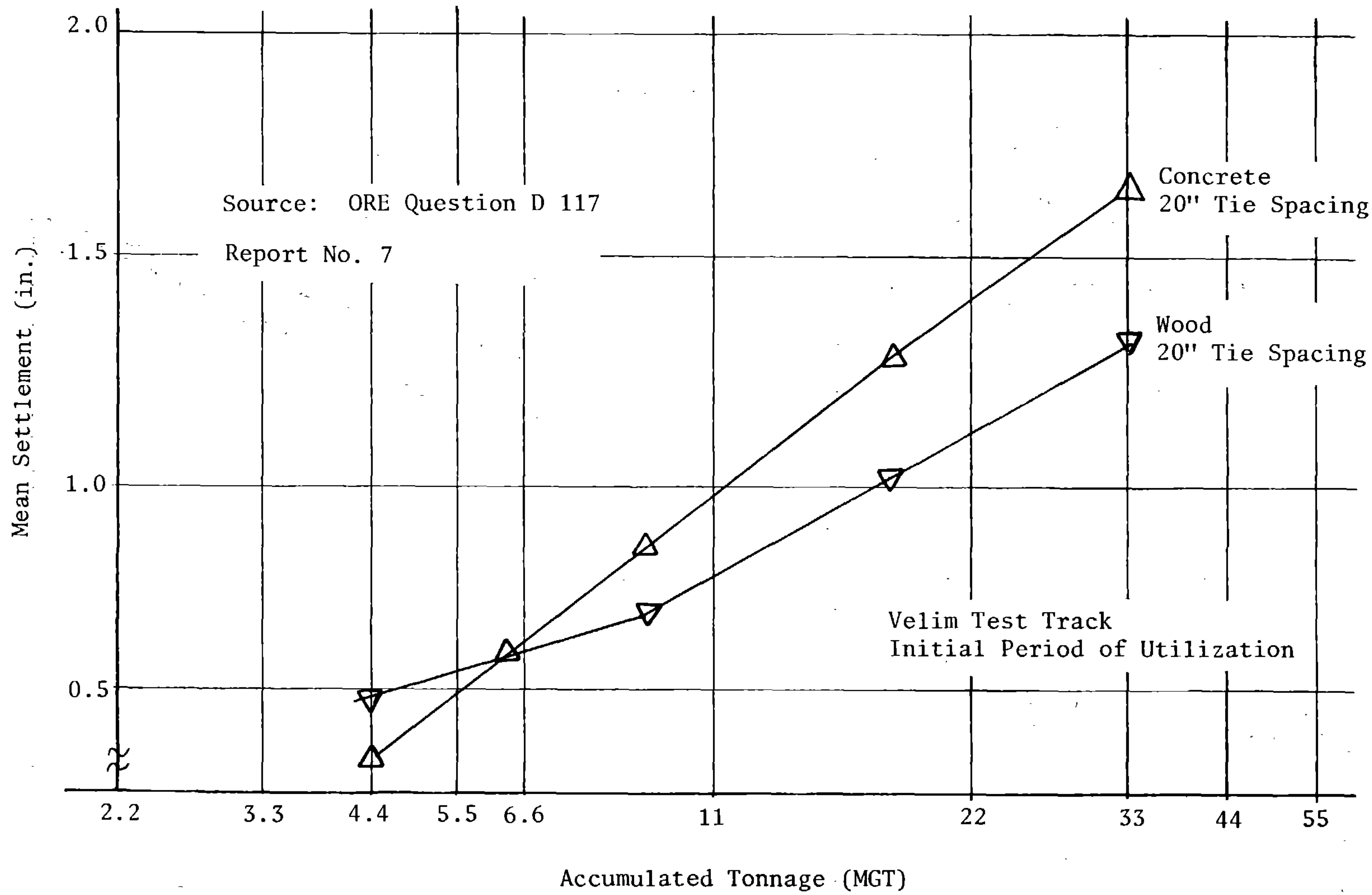


Figure 4-2. Settlement of Concrete and Wood Tie Track at Velim

equivalent track sections. Ballast at Velim consisted of approximately 12 inches of angular crushed stone in a newly constructed track.

Analysis by Committee D117 of the ORE on the behavior of the Velim test section indicated that, within the first 33 MGT of traffic, the newly constructed track has high settlement rates as shown in figure 4-3. During this early period, variations in construction play a greater role in influencing track settlements.

Figure 4-3 compares the mean settlement behavior of these track sections for 33 MGT following the initial "breakin" period. The concrete tie track shows a wide variation settlement in new construction applications, which is dependent on the spacing with somewhat better performance at closer tie spacings.

Potentially more significant, for applications to existing roadbeds, are the lower curves of figure 4-3 (identified by the triangular data points) which indicate the performance of concrete ties installed on existing roadbeds at Velim. Both installations show substantially lower rates of settlement as compared with new construction. The trends exhibited also depend greatly on the type of foundation material, and translation to other applications is, of course, dependent on the site specific characteristics of the roadbed.

The mean settlement behavior of track sections provides a general indication of track surface deterioration. However, mean or overall settlements are more significant in terms of their relationship to differential settlement. Differential settlement provides information on the changes in the relative elevation of the various points along the length of the track and consequently is interpretable as vertical track roughness. It is this roughness which is directly related to the ride quality and uniformity of the track surface.

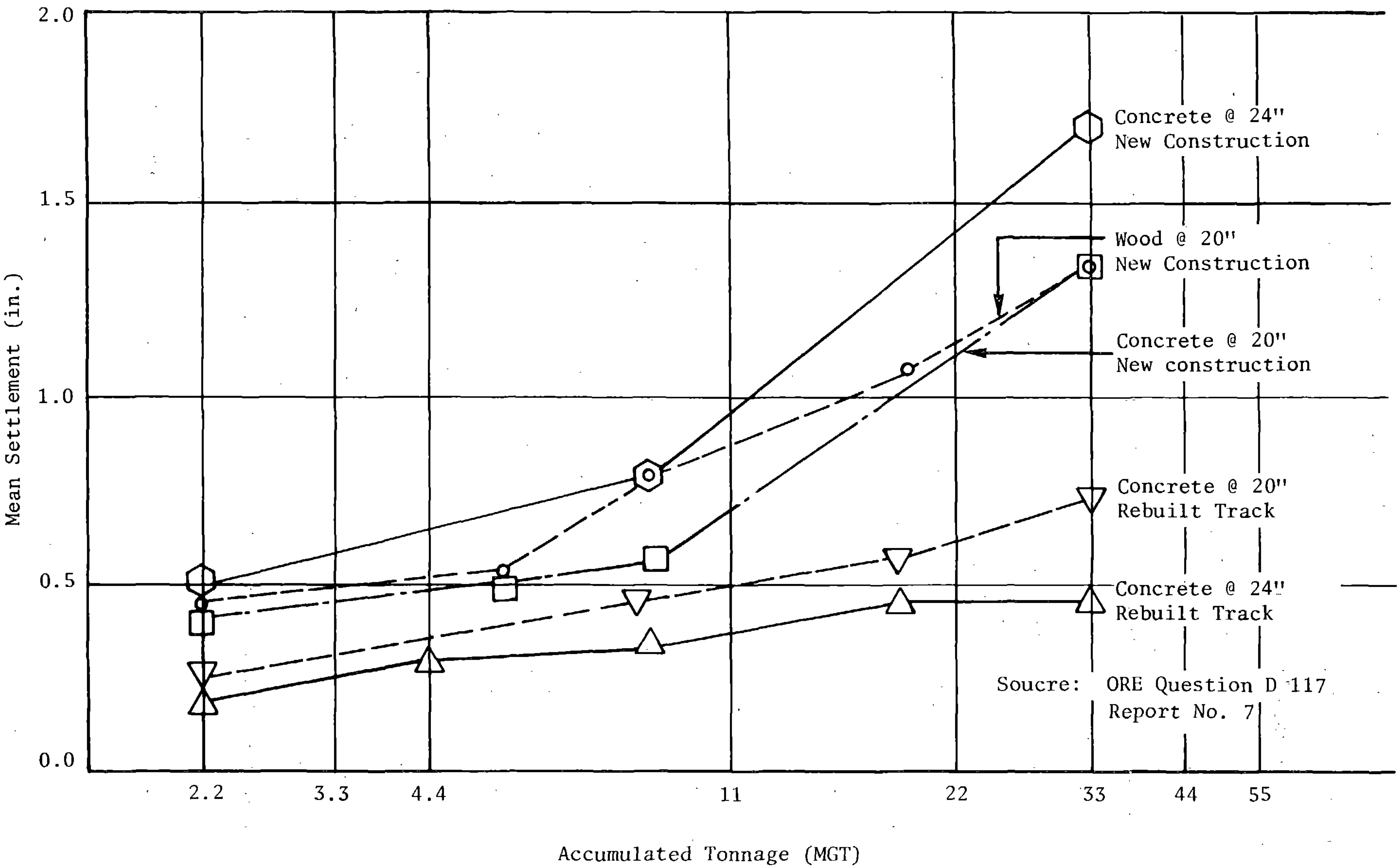


Figure 4-3. Change in Mean Settlement at the Velim Test Track

An index of track differential settlement at a point in time can be derived from an examination of the scatter of the vertical settlement along a given section of track. A simple measure of this scatter is the standard deviation of the track settlement data. The lower the value of this parameter, the more closely the data clusters about the mean and the more uniform the track surface.

The test data for the newly constructed track sections at Velim are compared (in figure 4-4) for a period from 2.2 through 33 MGT. The principal finding from this limited comparison is the more uniform settlement behavior of the concrete tie track as evidenced by the lower standard deviation of the data. This data covers comparable service with equivalent maintenance cycles. This data supports the contention that concrete tie track construction provides a more uniform track surface. Considering similar data for North American conditions, preliminary indications from the C&O/B&O Noble, Illinois concrete tie test track do not contradict this trend.[3]

Summarizing the general trends in track settlement data for wood and concrete tie track construction, three main points emerge. The mean settlement data for both the FAST and the Velim test tracks indicate that concrete tie track tends to settle more rapidly, initially, than wood tie track. This trend is most pronounced during the first 33 MGT of traffic for new track.

However, when installed as rebuilt track on existing roadbeds, concrete tie track structures showed a substantial decline in both accumulated long-term settlement as well as settlement rate. This suggests that for newly constructed concrete and wood tie tracks designed for equivalent service conditions, embankment and ballast preparation play an important role in governing overall track settlement. This aspect of new track construction may more strongly influence variations in settlement rate than tie type for equivalent track designs.

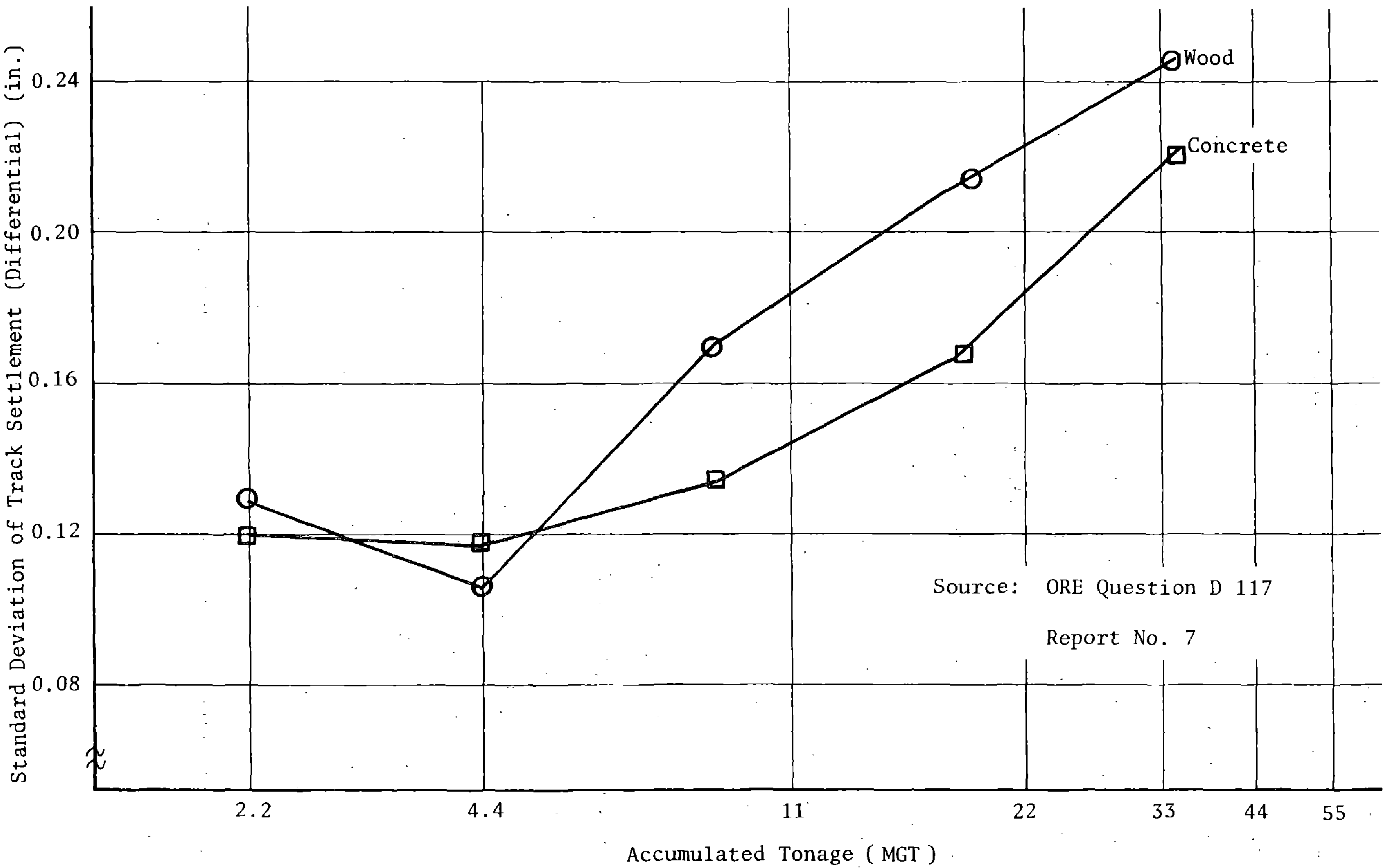


Figure 4-4. Change in Standard Deviation of Settlement at Velim Test Track - Index of Differential Settlement

Tie type appears to more strongly influence the uniformity of track settlement. Concrete tie track settlement data showed less scatter along the test track length than data for an equivalent wood tie construction. This trend is consistent with indications of concrete tie track behavior under normal revenue service.

This behavior should be reflected in track geometry data in terms of more uniform track profile and surface and should also be manifested in lower levels of twist errors. Continued observation of track behavior is required to substantiate this trend as neither FAST nor Velim track data were adequate for confirmation.

4.3 Track Geometry Deterioration With Traffic

The geometry of track section and the changes of track geometry with accumulated traffic represents a readily interpretable measure of both the adequacy of the track at any instant for a given level of service and of the long-term ability of the track to provide that service without excessive deterioration. Consequently, various measures of track geometry have been employed to monitor track quality, most tailored to specific applications.[4 5] Geometry data offers the advantage of providing a readily interpretable measure of track quality coupled with the ability to rapidly accumulate data through the use of an instrumented track geometry car.

Data generated by a track geometry car differs in a fundamental sense from the survey to bench mark data discussed previously. The primary difference lies in the reference or datum from which track geometry deviations are taken. Most track geometry cars, and in particular, the cars which have been employed at FAST, utilize the concept of determining the deviation of the rail from the midpoint of a (real or numercially constructed) chord. Therefore, track geometry car data represents variation of the rail position with respect to a moving reference system. It represents a measure of the position of the rail, relative to two adjacent locations on the rail which define the position of the chord. Most often this chord is taken as 62 feet. While providing a measure of geometry errors, a single chordal measurement system does not provide an accurate characterization of the spatial distribution of geometry errors. A more complete discussion of these problems is given by Yang.[6] Likewise, survey to benchmark data is for an unloaded track while the car produces a track deflection for crosslevel and twist, and little or no deflection for other data.

The problems in interpreting geometry data from FAST is further complicated by conditions specific to FAST, most noticeably the startup and shakedown problems which were encountered with the geometry acquisition and processing systems. Due to the demands of the FAST startup schedule, a series of geometry cars have been employed. During the period through 16 MGT, a loaned car built to European standards was employed. Data from this car, while adequate for maintenance surveillance purposes, do not appear to exhibit the levels of accuracy or repeatability required to facilitate direct processing for research purposes. Beyond 16 MGT, the Test Center began operation of a similar self-contained chordal measurement car. This car has suffered a series of shakedown problems and is just now (60 MGT) approaching full operation. Consequently, reliable track geometry data for the first 45 MGT of FAST operations, the period with which this report deals, is available for relatively few points in time with reliable track profile and surface data being virtually nonexistent.

In addition, constraints imposed by the online geometry data reduction cabilities for FAST existing during this initial or startup period, have imposed limitations on the scope of the data analysis which could be performed.

Especially significant was the limited ability to discriminate during data processing between curve and spiral data in characterizing track alinement. As figure 4-5 illustrates, this problem in data processing would cause even well-alined track segments containing both spiral and constant curvature sections to show a wide scatter of alinement error data. This problem arises from the interpretation of the midchord offsets for properly alined spirals as errors in the data for the constant radius curve. While this problem could be overcome by processing the data either to separate out the spiral data or to subtract the theoretical midchord offset at each data point, the constraints of time and the limitations of the existing software available to process the FAST data precluded the use of this technique.

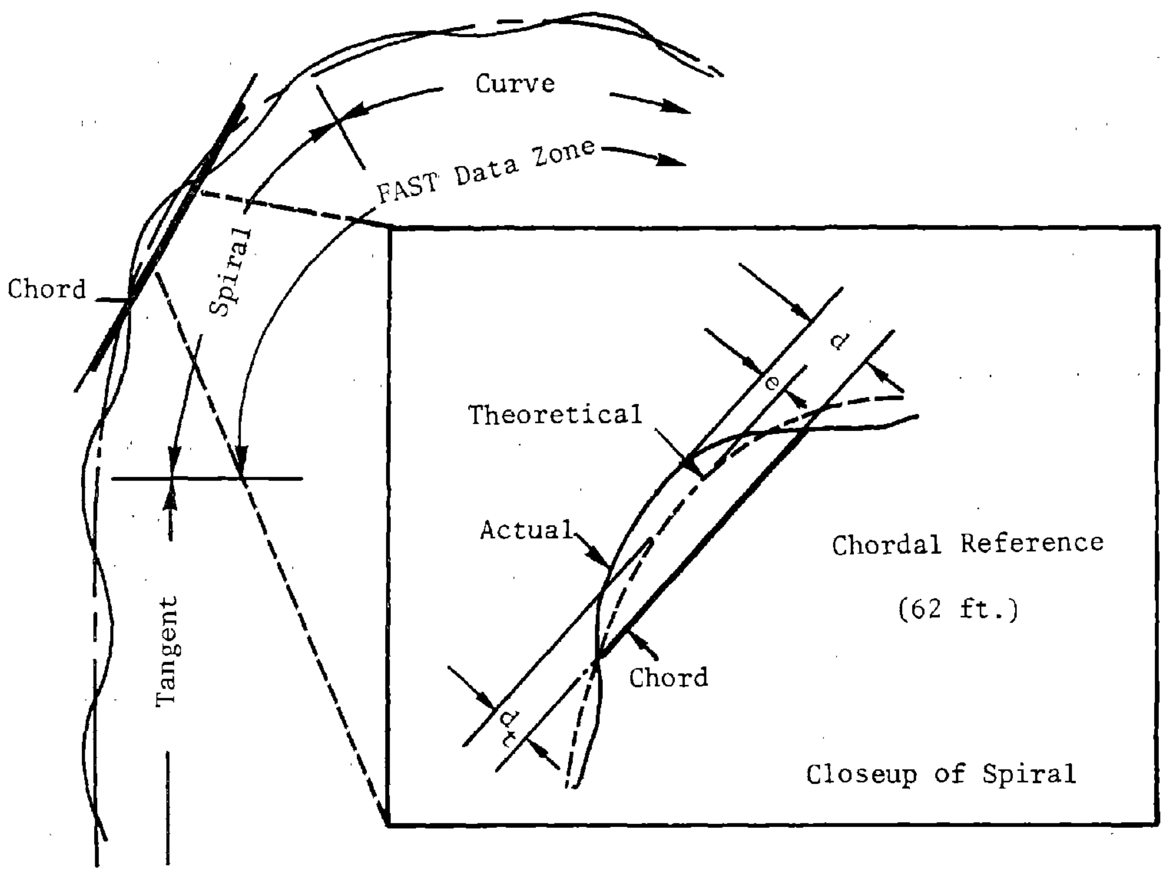
In order to overcome this problem with chordal data, two indices of track geometry retention were defined which are independent of these limitations in the data processing systems. Assuming that the scatter in the alinement error data due to the theoretical track line is fixed for any segment of track, track shift and track roughness indices were defined as the ratio of the mean alinement error data for any given tonnage to the mean error at a base period near the startup date for FAST. That is:

$$\text{Track Shift Index} = T = \frac{\text{Mean of chordal offsets at given MGT}}{\text{Mean of chordal offsets at base MGT}}$$

$$\text{Track Roughness Index} = R = \frac{\text{Standard deviation of chordal offsets at given MGT}}{\text{Standard deviation of chordal offsets at base MGT}}$$

For both of these indices, a value of 1.0 would represent no change in track alinement with tonnage. Similarly, a value greater than 1.0 signified deterioration of track geometry while a value less than 1.0 indicates improvement compared with the base period.

Figures 4-6 and 4-7 represent the comparison of track shift and roughness indices for the wood and concrete tie test sections of FAST for the 5^{o} (nominal)



$$d_t + e = d$$

Theoretical Midchord Offset +

Midchord Offset of Alinement Error =

Total Measured Midchord Offset

- FOR FIXED RADIUS CURVE: d_t is known and may be subtracted from recorded values
- FOR A COMBINATION OF CURVES AND SPIRALS:

  d_t is unknown and mean shows shift due to variation of d_t throughout the spiral
- HOWEVER, THE SHIFT IN THE MEAN DUE TO THIS FACTOR IS FIXED FOR A GIVEN TRACK FOR ALL TIME

Figure 4-5. Track Geometry Data Acquisition at FAST

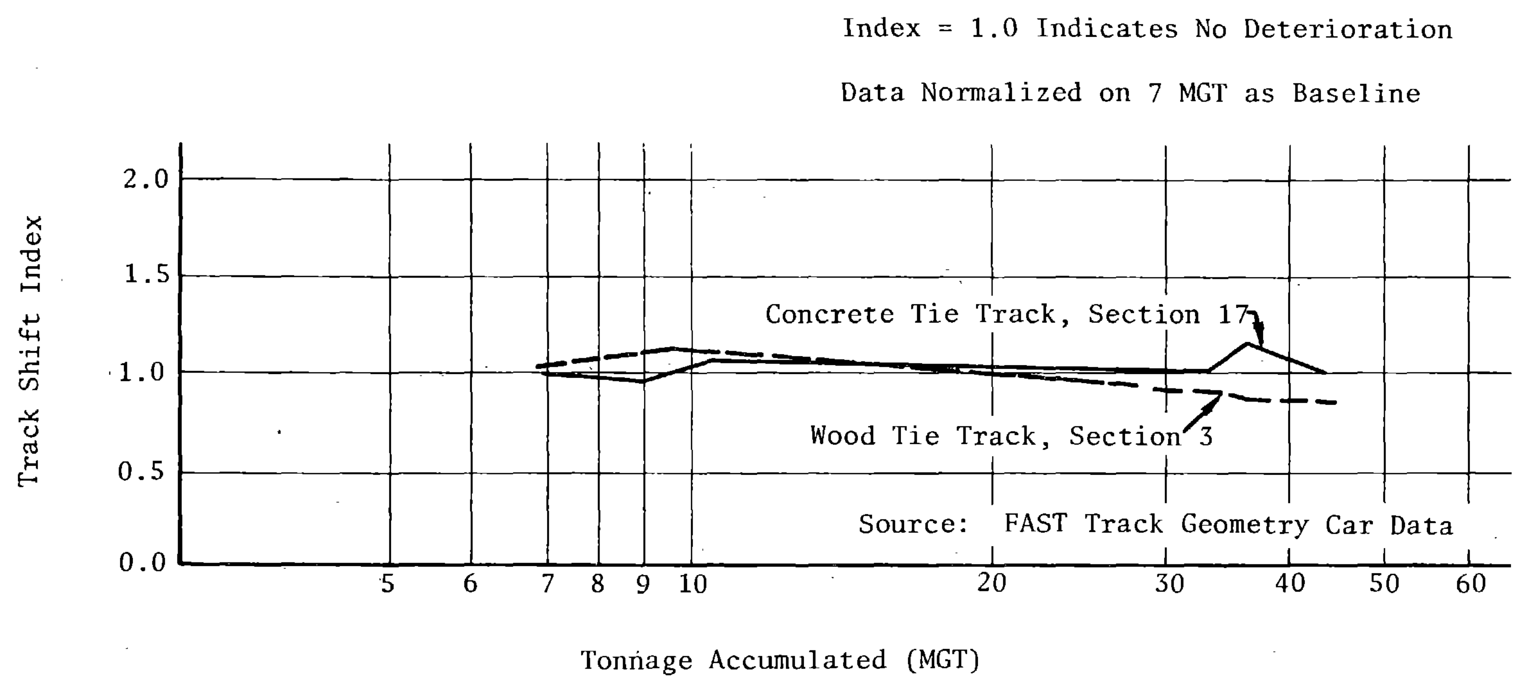


Figure 4-6. Mean Alinement Error for 5° Curves

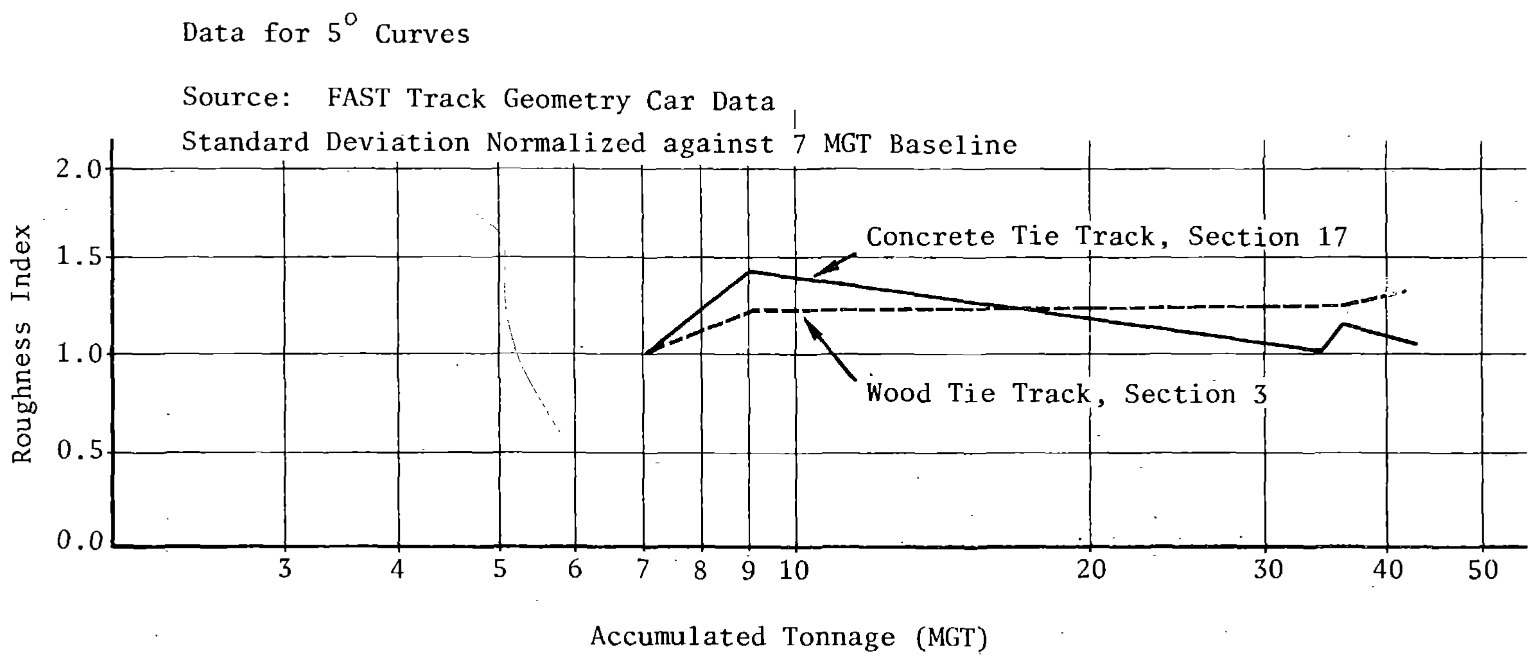


Figure 4-7. Track Roughness Index for FAST Alinement Data

curves in sections 3 and 17, respectively. The principal feature of this data is the small variation of the indices with tonnage and the apparent improvement with tonnage accumulation. Analysis of the raw data and visual track inspection data indicates that these trends are most directly attributable to improvements in the track geometry acquisition system. FAST track geometry car data consequently is of very limited applicability in determining the relative effectiveness of FAST concrete versus wood tie track in maintaining line during the first 50 MGT.

Turning to European data, also developed from a chordal measurement system, defining comparable service conditions for developing comparisons of alinement retention capability proved to be difficult. Most directly comparable are the concrete and wood tie track data for the Cologne-Weisbaden line[7] presented in figure 4-8. The most significant trend in this data is the more gradual rate of deterioration of track alinement for the concrete tie track as compared with the wood tie track for equivalent maintenance cycles. This trend was reported by Janin[8] as typical of the behavior of these two systems. It should also be noted that the concrete tie track, while showing a slower rate of alinement decay, also shows greater mean errors through the period of 33 MGT. Janin attributed this behavior to the lower relative accuracy of track lining operations when dealing with the substantially higher weight of the concrete tie.

The service environment for the Cologne-Weisbaden line is within the bounds of passenger service with mean axle loads of from 8 to 14 metric tons. In order to interpret the behavior of these two track constructions at the higher axle loads indicative of North American freight service, data on track geometry behavior for comparable wood and concrete tie track on the Florida East Coast Railway Company (FEC) is compiled in table 4-3. Compiling the scatter in profile, alinement, and crosslevel data, as measured by its standard deviation, in all

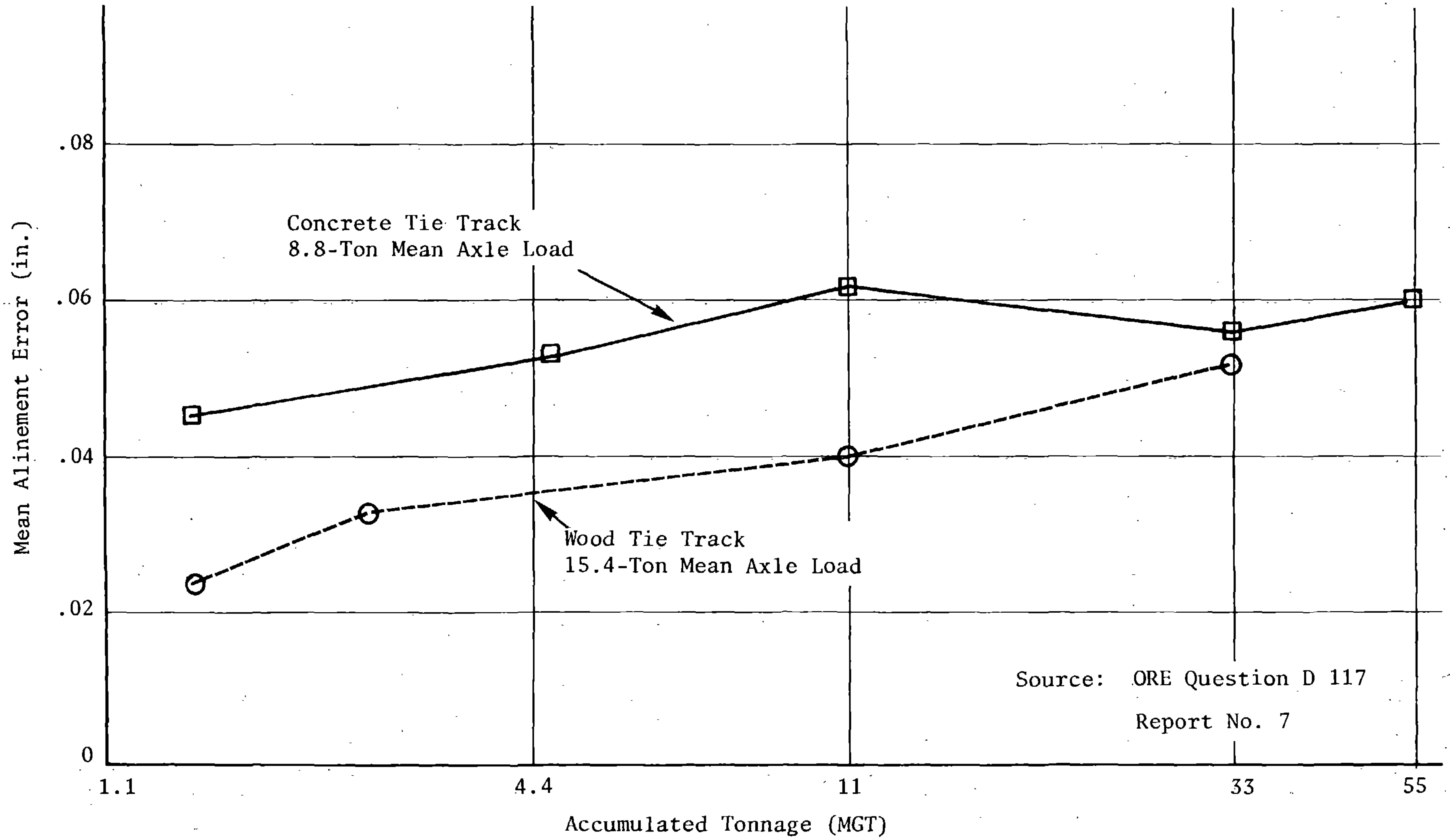


Figure 4-8. Mean Alinement Error for Cologne - Wiesbaden Line

Table 4-3. Summary of Track Geometry Data for Wood and Concrete Tie Track of Florida East Coast Railway Company

Track Description	Mile Post	Standard Deviation (in.) of Track Geometry Irregularities		
		Vertical Profile	Cross Level	Alinement
A. Wood Tie Track (20-inch tie spacing)				
1. Recently surfaced	MJ10.0 - MJ13.0	0.22	0.15	0.15
2. 1-year service	170 - 173	0.30	0.22	0.15
3. 3-years' service-surfacing scheduled	191 - 194	0.24	0.27	0.18
4. 3-years' service-surfacing scheduled	91 - 94	0.21	0.26	0.22
B. Concrete Tie Track (24-inch tie spacing)				
1. Recently surfaced	263 - 266	0.51	0.11	0.14
2. Recently surfaced	177 - 180	0.11	0.09	0.12
3. 1/2-year service	30 - 33	0.23	0.12	0.18
4. 1-year service	12 - 15	0.31	0.09	0.18
5. 1-year service	277.9 - 278.25	0.22	0.09	0.15
6. 1-year after surfacing	202 - 205	0.14	0.11	0.15
7. 3-years' service - surfacing scheduled	115.8 - 118.8	0.19	0.11	0.12

cases, the concrete tie track shows lower values of this index. Particularly interesting are comparisons of crosslevel in items A2 and B5 (0.22 for wood versus 0.09 for concrete) and items A4 and B7 for alinement (0.22 for wood versus 0.12 for concrete).

This data indicates similar trends to the European data with crosslevel errors and alinement variation showing much smaller variations with time on concrete tie track when compared to the wood tie track. Profile data shows some difference in the performance of the two track systems with the exception of the data groups B1 and B4, in the concrete tie data. The FEC data is consistent with trends on profile, crosslevel, and alinement reports for the loop track at Velim.

Gage data, since it is measured against a known standard (56.5 inches), does not require the normalization discussed earlier, to compare chordal geometry data. Consequently, directly calculated values of mean gage error for wood and concrete tie track for both the 5^{o} curves and tangent track are presented in figures 4-9 and 4-10, respectively. Data from hand measurements of concrete tie track gage are included for comparison (also see figure 5-7). The data have not been corrected for gage point wear, which in this case is less for section 3, the wood tie track (see figure 6-1), because the track geometry data was taken throughout the section, including some rail types which have worn much less than the standard rail in section 17.

The trends which are evolving for tangent track (see figure 4-10) show that with increasing tonnage there is no corresponding increase or decrease in the difference in mean gage error between wood tie and concrete tie systems.

However, the gage data for the curves (see figure 4-9) does indicate a small difference in the rate of increase in the mean of the track geometry car data with, as might be expected, a slightly greater rate of increase in gage widening on concrete ties than wood ties. This is not a surprising trend considering

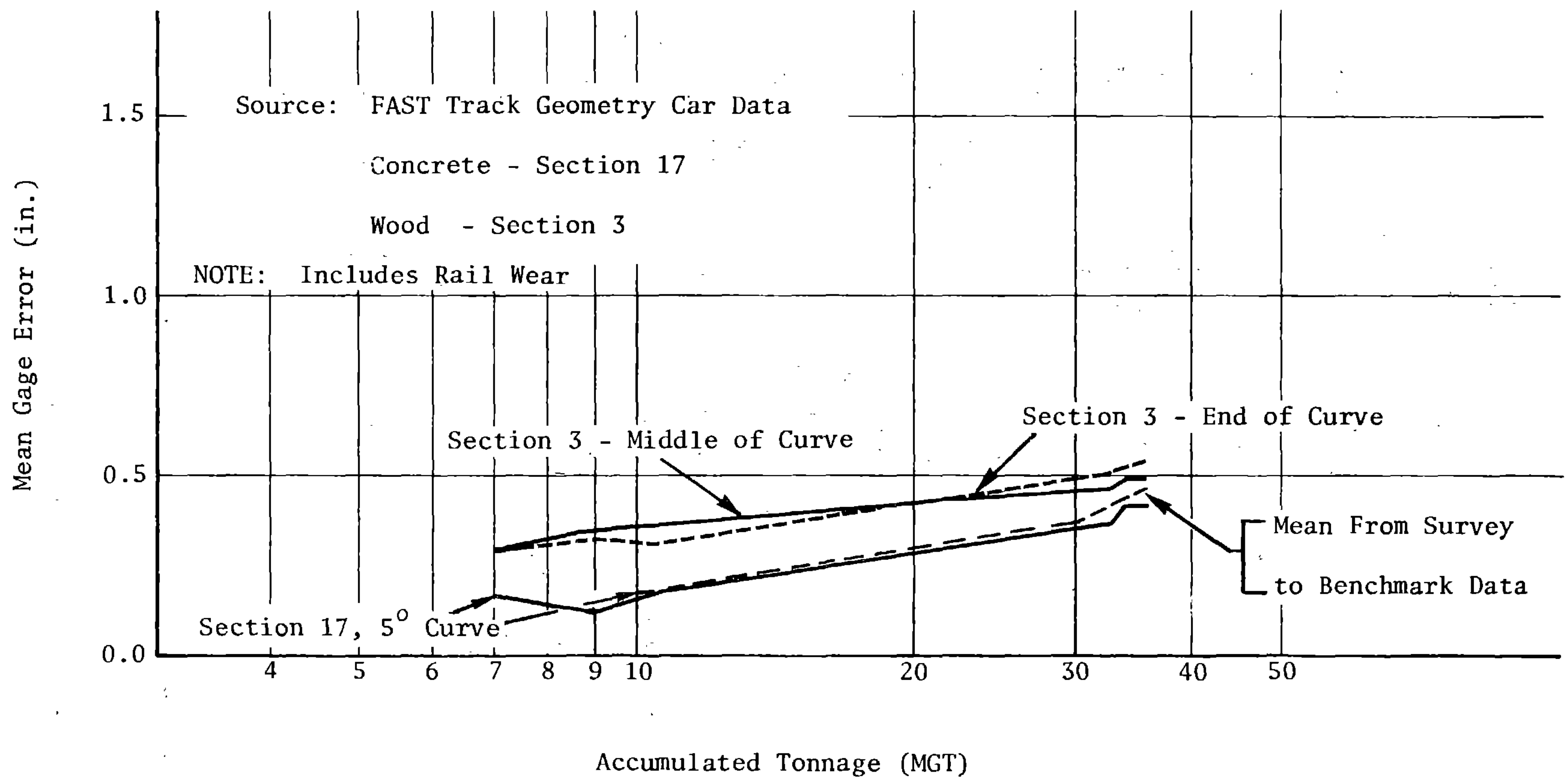


Figure 4-9. Comparison of Mean Gage Error on 5° Curves - Concrete vs. Wood

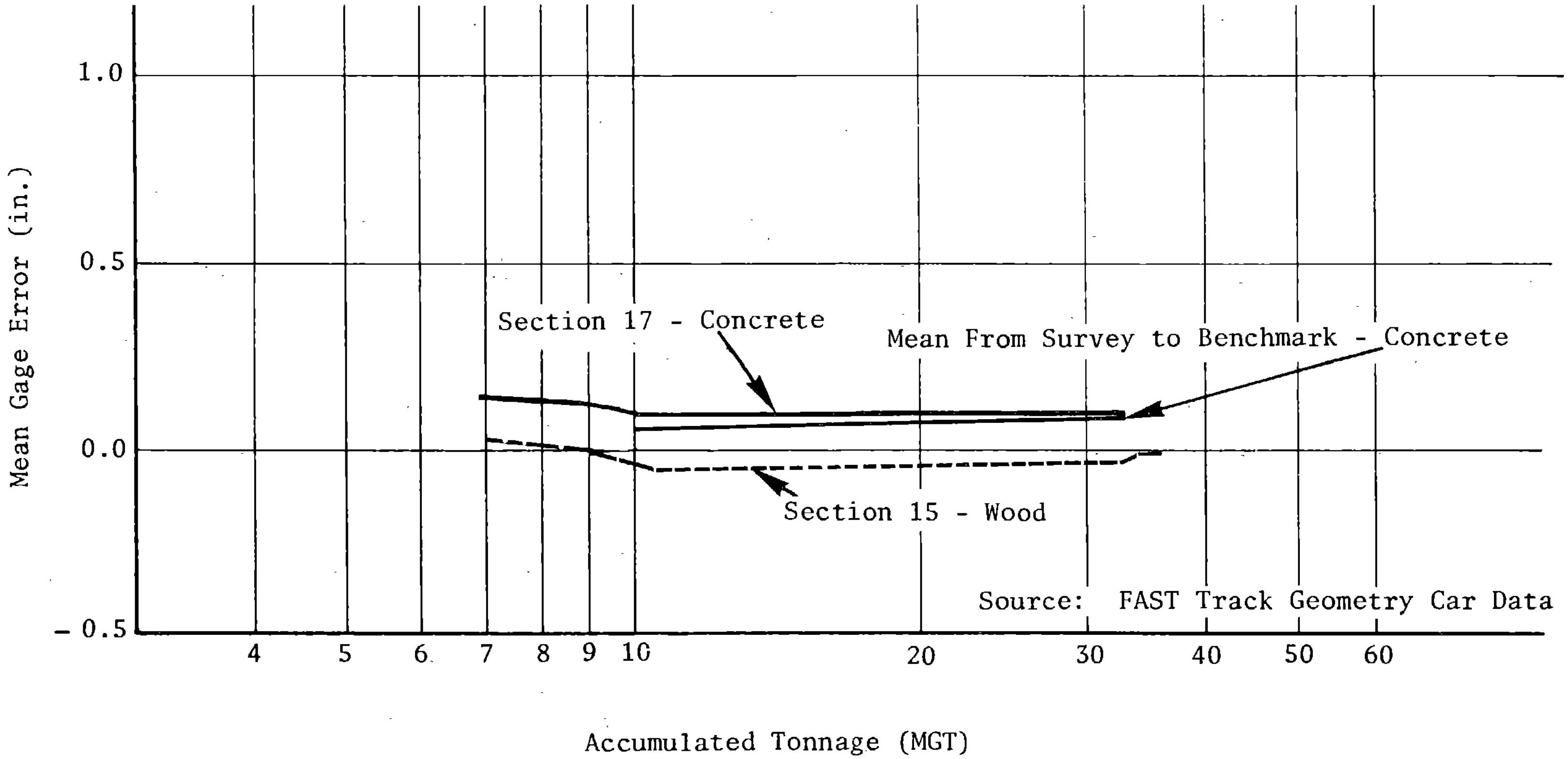


Figure 4-10. Comparison of Mean Gage Error on Tangent - Concrete vs. Wood

that the rail wear values have not been removed from the track geometry data. When the wear data (see figures 6-1 and 6-3) are removed from the data, the relative rate of increase in gage is slightly greater on wood ties than on concrete ties. This trend showing better gage retention is consistent with verbal reports from virtually all other concrete tie installations, although no hard data was available to confirm these comments.

In summarizing the results of the track geometry data analysis, it should be noted that the geometry data acquisition and reduction systems at FAST are continuing to undergo shakedown and modification. Consequently, the track geometry data from FAST shows substantial variation due largely to changes in the capability of the data acquisition system.

Alternative sources of track geometry data were analyzed including foreign and domestic railroad data. The general findings are that concrete tie track shows a lower rate of geometry deterioration with traffic than a comparable wood tie track. This was especially true for alinement and crosslevel errors. However, the data also indicates a strong dependence of concrete tie track on the accuracy of track lining equipment in establishing an initially uniform track. Failure to accurately line and surface the track results in errors of greater magnitude which fortunately do not grow as rapidly as on wood tie track. This suggests that realization of the full potential of the concrete tie places a premium on the ability of maintenance equipment to handle the significantly heavier track.

Data available from FAST indicates slightly better gage retention for track employing concrete ties. However, the benefits of concrete tie track in reducing rail wear could not be accurately determined from the FAST data available through 45 MGT. Canadian National results on 6^{o} curves at Jasper indicate almost double the rail life as compared to rail on wood ties.

4.4 Vertical and Horizontal Track Resiliency

Track settlement and geometry data are direct measures of the condition of the track and offer readily interpretable measures of track performance. However, they are only indirect measures of the response of the track to train loads, representing the accumulated deterioration of the initial alinement of the track structure.

An indication of the structural characteristics of the track system and its response to wheel loads can be derived from an analysis of the deflection of the track structure under vertical and lateral loads. These deflections ultimately can be related to the stress conditions induced in many of the structural components of the track superstructure (rails and ties) as well as in the foundation (ballast and roadbed).

As early as 1867, it was realized that the deflections of the track under load could be well represented by analyzing the track as a continuous beam of unlimited length, resting on an elastic foundation.[9] In this analysis, all of the structural characteristics of the rails, ties, and fasteners are lumped into the properties of a single "track beam."

Similarly, the properties of the ballast and roadbed are lumped into a foundation model which represents this complex substructure as a bed of elastic springs. The sole parameter which characterizes the properties of this continuous elastic foundation is the spring constant, most often referred to as the track modulus. Since, with some modification, this theory is equally applicable to the response of the track both vertically and horizontally, two track parameters, namely the vertical and horizontal track moduli are required to characterize the support conditions offered the track superstructure.

In practice, moduli values are determined by monitoring the deflection of the track under known loads and then computing the moduli values. This process was carried out on the FAST track by analyzing the deflections produced by a

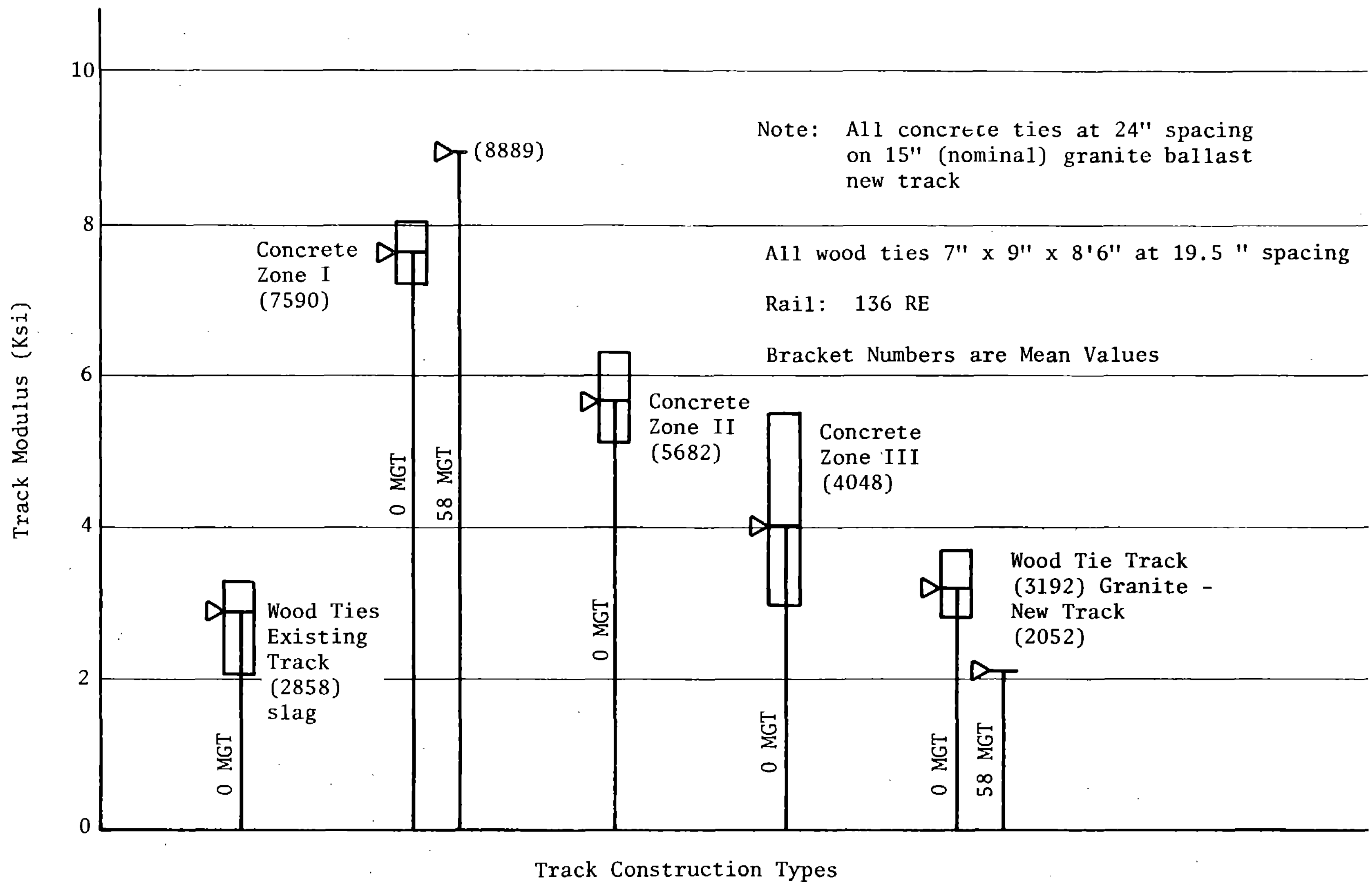


Figure 4-11. Comparison of Vertical Track Modulus at FAST Track -

Primary Variable: Tie Type

locomotive truck and calculating moduli values. Figure 4-11 presents the results of this analysis when applied to track constructions employing three types of concrete ties and two wood tie sections. Both the range and mean values for the moduli are shown. All the track was constructed with 136RE rail and granite ballast (15-inches deep) except one section in the wood tie segment which was slag ballast.

As this figure illustrates, the concrete tie track exhibited substantially higher vertical track moduli through the limited service history represented on the figure. As figure 4-12 indicates, the type of tie employed influences track moduli more strongly than the type or depth of ballast, largely due to the higher rigidity of concrete ties and their greater effective bearing area when compared to closer spaced wood ties.

The implications of these moduli values can be determined from figures 4-13 and 4-14. The first of these presents the maximum rail base bending stress for a variety of rails under a static locomotive load. The influence of an increase in track modulus is quite pronounced and more strongly influences the maximum rail base stress than does rail size over the range of rails nominally available (47 percent variation versus 27 percent variation, respectively). It should also be noted that beyond a modulus value of 3,000 psi, the curves flatten rapidly with increasing moduli, indicating that no real gains relative to rail stress are realized at appreciably higher moduli values. Almost all FAST data falls in the upper bracket of this range of values, implying lower rail flexural stresses.

Similarly, figure 4-14 represents the influence of track moduli on maximum rail deflection under the same loading condition. As can be seen, rail deflections decline quite rapidly with increasing moduli values, while rail size variations produce only minimal changes in deflection. Track modulus then is

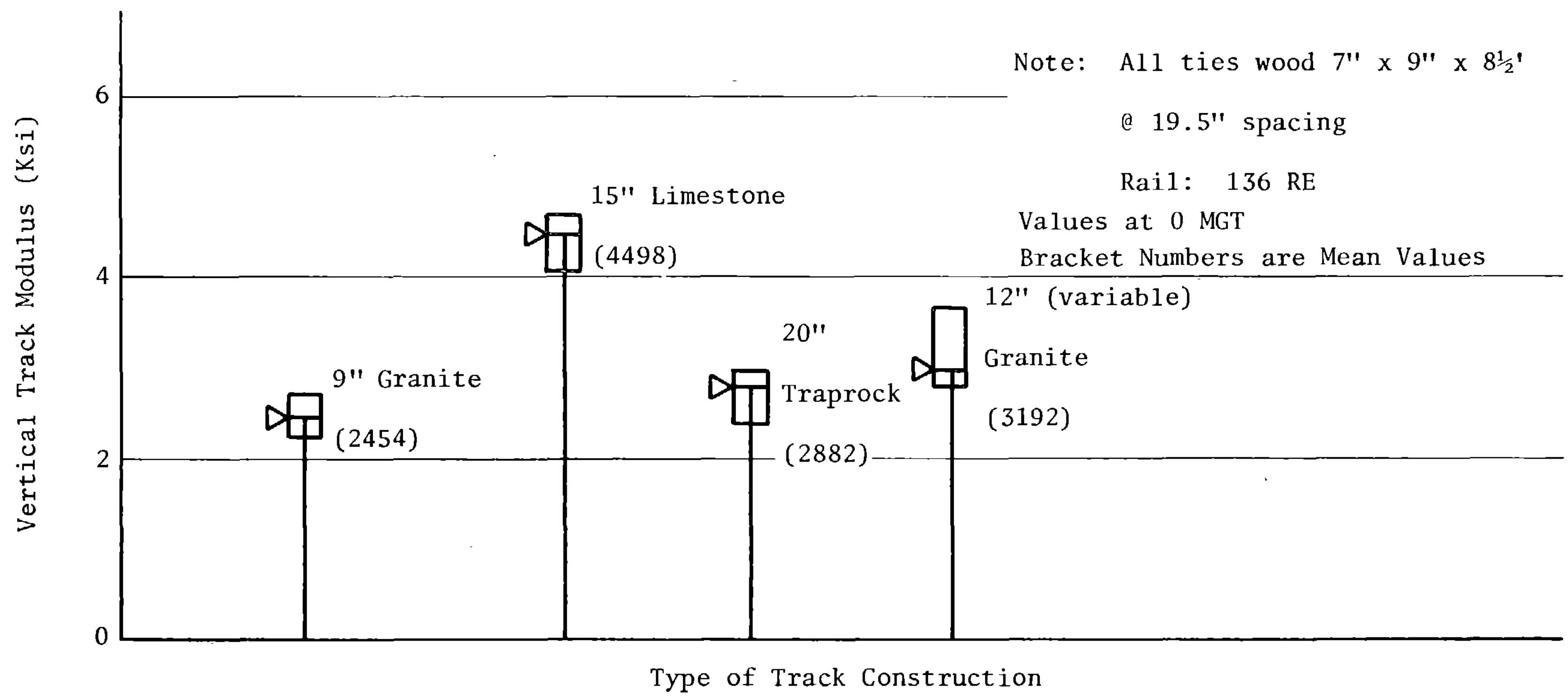


Figure 4-12. Comparison of Vertical Track Modulus at FAST Track - Primary Variable: Ballast Properties

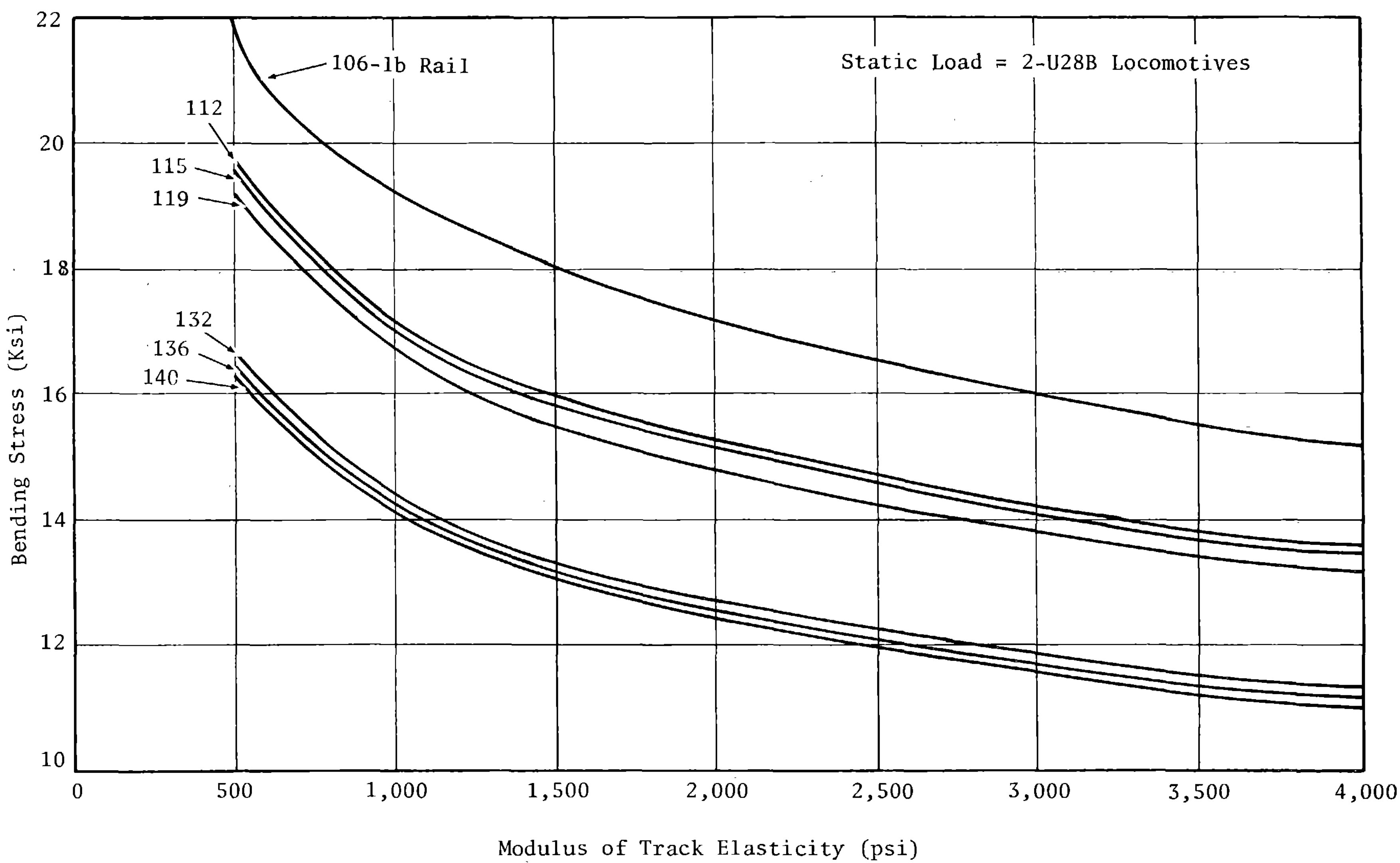


Figure 4-13. Effect of Track Modulus on Maximum Bending Stress for Various Rail Sizes

After Butler*

*A.B. Butler, "An Analysis of Bending Stresses and Deflections in Railroad Rails," Master Thesis, University of Illinois, Urbana, 1969.

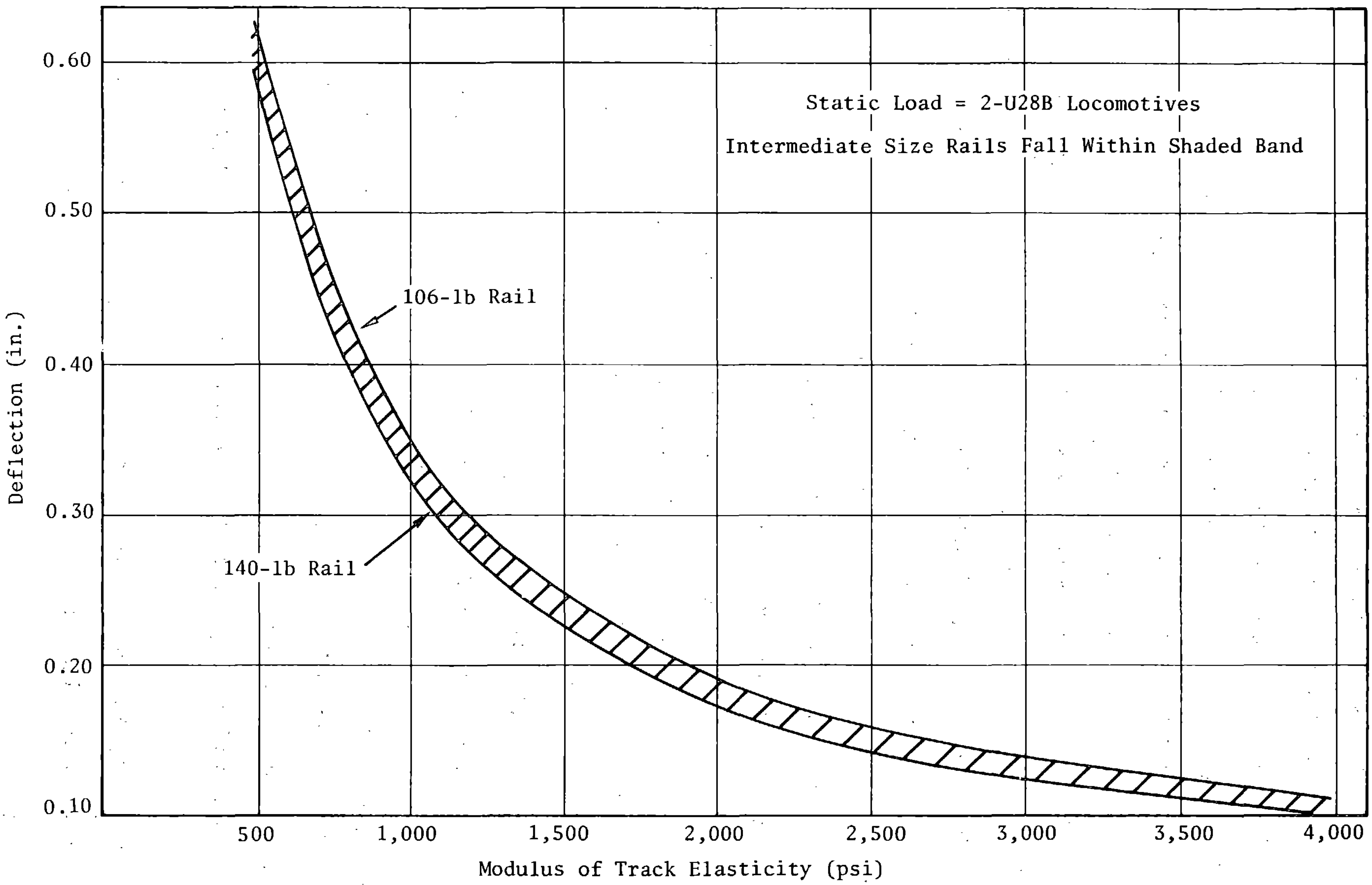


Figure 4-14. Effect of Track Modulus on Track Deflection for Various Rail Sizes After Butler*

*A.B. Butler, "An Analysis of Bending Stresses and Deflections in Railroad Rails," Master Thesis, University of Illinois, Urbana, 1969.

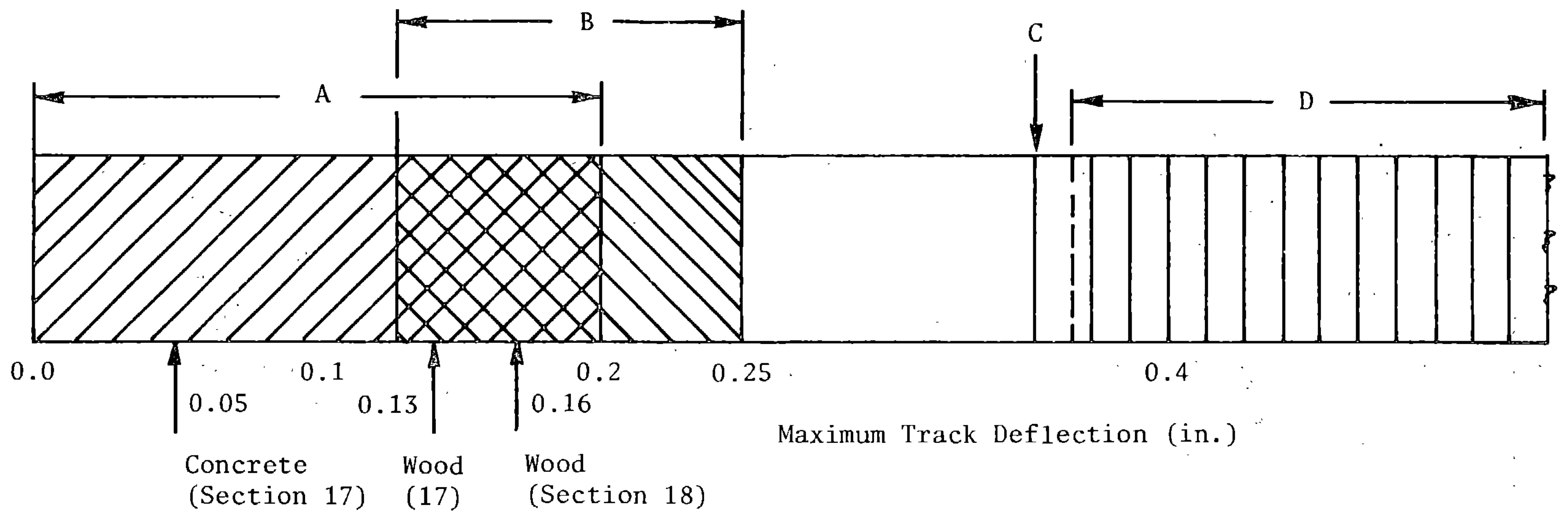
the dominant feature controlling the maximum track deflection under load and rail bending stress for a given loading condition.

It has been postulated by both Jenkins[10] and Lundgren and Hay[11] that maximum track deflection can be directly related to the geometry retention ability of the track. Lundgren and Hay have proposed the evaluation scale shown in figure 4-15 to predict the rate of track deterioration based on maximum rail deflection. Superimposing maximum rail deflections at FAST on this scale shows the predictions of track behavior due to the high moduli. It should be noted that the concrete ties in zone I, tangent track, show an extremely high track modulus. In curved regions with high moduli values, penalties in terms of contact stress damage such as reail corrugations may become significant.[12]

In addition to the vertical moduli tests conducted at FAST, a simple test to determine a lateral modulus was conducted for a limited number of track locations. In this test, equal lateral loads with no vertical load were applied to one rail base at a 5-foot spacing to simulate lateral loading due to a freight car truck. The test load procedure is identical to that employed on the C&O/B&O railroad during the ballast consolidator tests.[13]

Figure 4-16 illustrates the substantially different response of concrete tie track subjected to lateral loads when compared to conventional wood tie construction. Concrete tie track shows a much more uniform deflection, most likely arising from the increased rigidity of the track panels employing elastic fasteners. This mode of deflection brings the ties adjacent to the loads into play more effectively in resisting track shift, and it is reflected in the much lower deflections evidenced in figure 4-16 and the much stiffer response shown in figure 4-17.

The response, in terms of maximum lateral rail deflection under load, shown in figure 4-17, has two significant features. First, the track modulus value



Range	Track Behavior
A	Deflection range for very low settlement rate
B	Normal desired maximum deflection for heavy track to give desired combination of resiliency and stiffness
C	Limit of desirable deflection for light track (100 RE and below)
D	Poor track yielding a high settlement rate

Figure 4-15. Performance Prediction for FAST Track Based on Lundgren and Hay Evaluation Scale

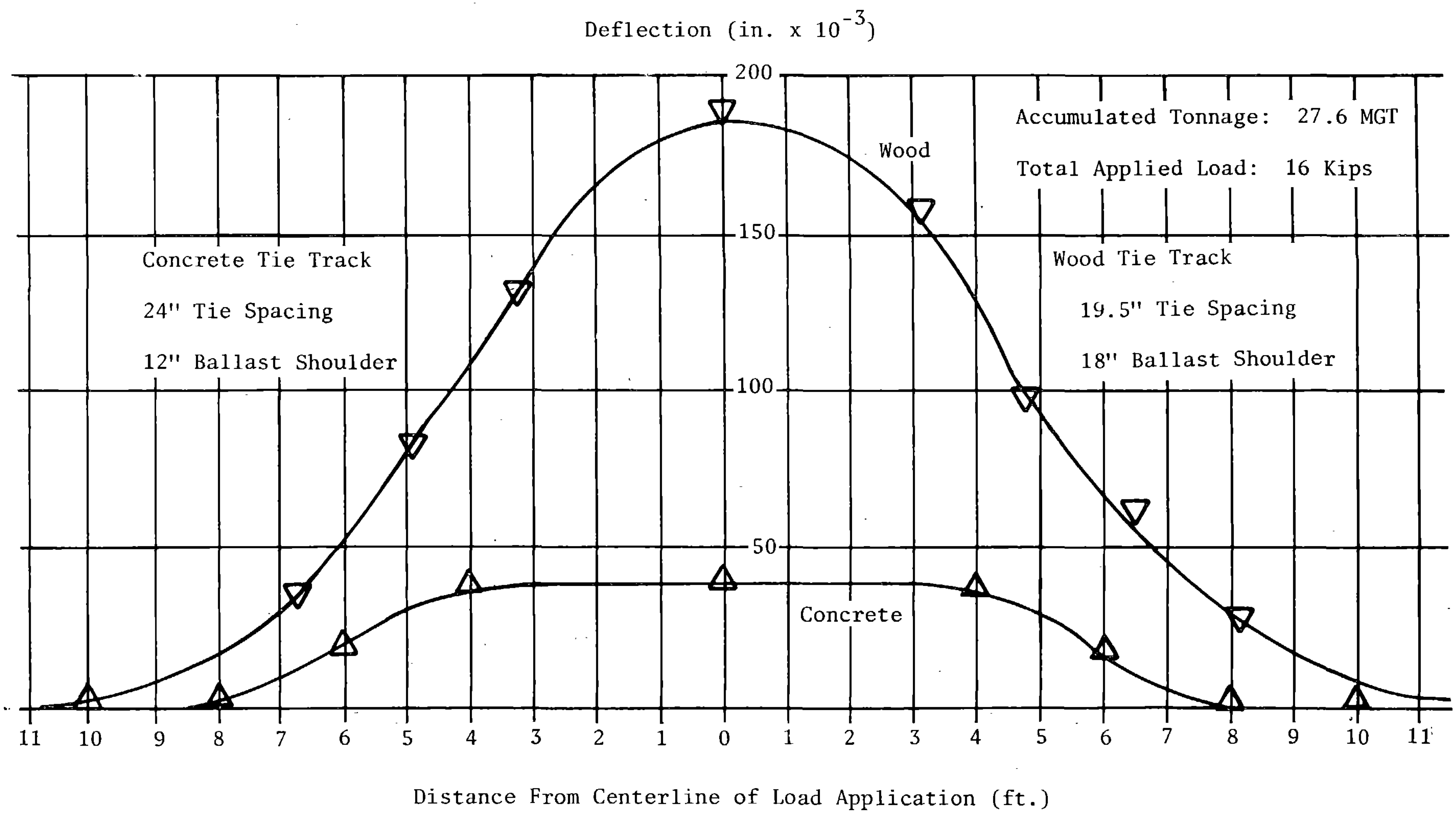


Figure 4-16. Track Horizontal Deflections - Wood vs. Concrete Ties

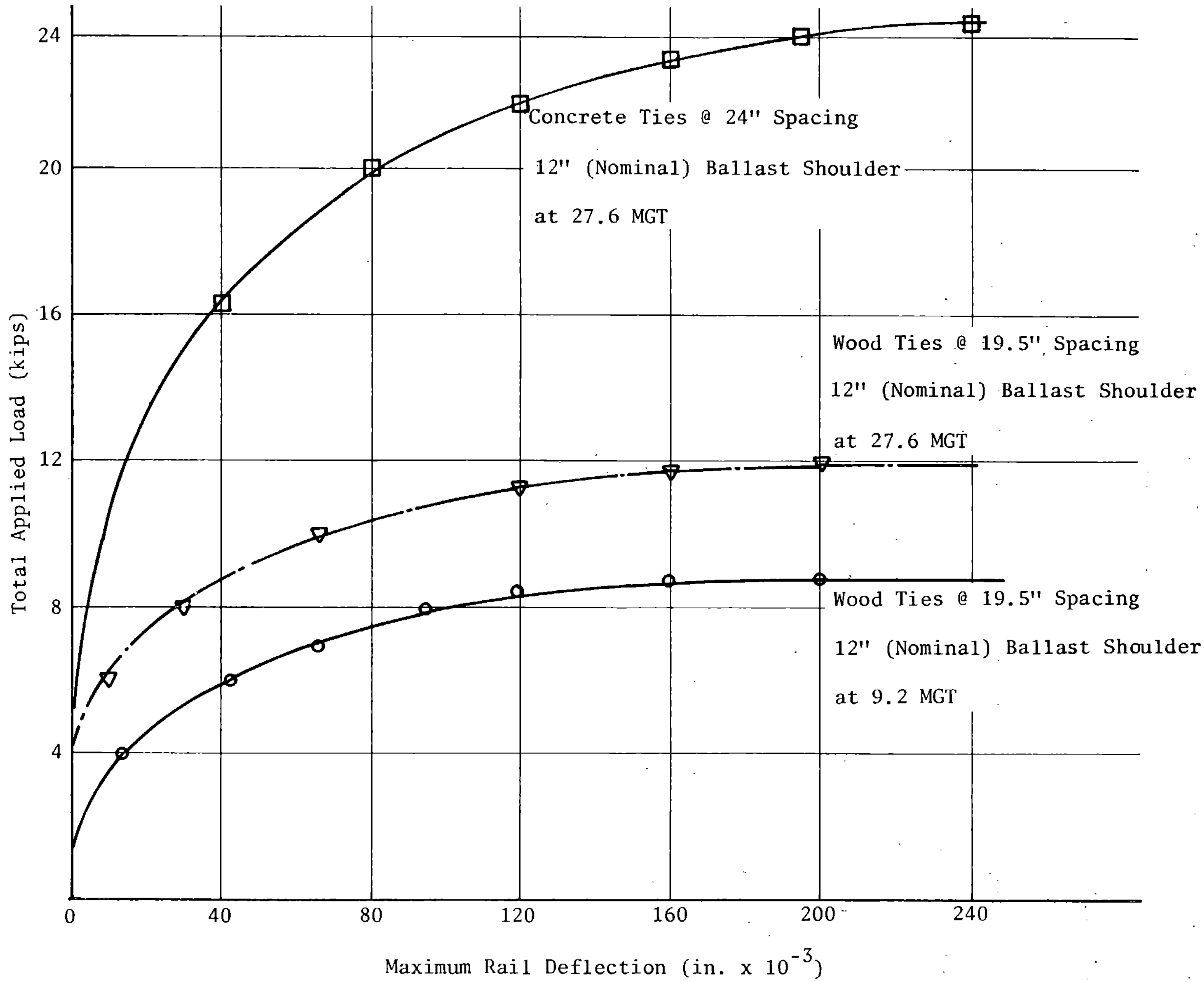


Figure 4-17. Load vs. Rail Deflection for Concrete and Wood Tie Track

represents a measure of the resiliency of the track under traffic induced loads. It represents the slope of the load-deflection curve.

Second, the load at yield (the point at which the curves in figure 4-17 become horizontal) is a factor most directly related to the stability of the track under logitudinal loads, i.e., track buckling. As table 4-4 summarizes, concrete tie track demonstrates better capability against both of these indices. This is also true in the data from the C&O/B&O test shown in table 4-5.

In summary, concrete tie track exhibits substantially higher moduli values, both vertically and horizontally, when compared with equivalent wood tie construction. These values should be manifested both in lower rail bending stresses and track deflection under load. This, in turn, suggests greater geometry retention capabilities for concrete tie track construction.

Equally important, concrete tie track demonstrates a high lateral yield strength. This factor is crucial in determining the stability of continuously welded rail as well as to the retention of track alinement. In both categories, concrete tie track should possess a substantial advantage based on the admittedly limited data sample.

Table 4-4. FAST Horizontal Track Stiffness

Test Section	Tie Type	Section History	Ballast Shoulder (in.)	Tie Spacing (in.)	Tonnage (MGT)	Stiffness † @ 10 kips (psi)	Load at Yield (kips)
17	Wood	New	12	19.5	9.2	*	8.8
17	Wood	New	12	19.5	27.6	1260	12.0
17	Concrete	New	12	24	27.6	9575	24.0
15	Wood	Existing	6	19.5	28.0	450	12.4
15	Wood	Existing	18	19.5	27.6	1300	17.2

Rail Size: 136 RE

†Stiffness computed from Beam on Elastic Foundation Theory with track rigidity = 2x rail rigidity

*Track yielded below 10 kips

Table 4-5. Summary of Horizontal Stiffness Tests

on the C&O/B&O Railroad after Cunney*

Summary of Lateral Forces and Corresponding Displacements

Test	Displacement (in.) at Load (lbs)				Force (kips)		Traffic
Panel	12,000	15,000	Yield	Maximum	Yield	Maximum	(MGT)
1 Wood	1.74	-	1.74	2.29	12.25	12.25	0
2 Wood	0.59	-	1.53	3.00	14.0	14.4	0
3 Conc	0.70	1.88	1.88	2.12	15.1	15.0	0
4 Wood C	0.36	-	0.63	2.31	14.3	15.5	0
5 Wood C	0.29	0.94	0.94	1.82	15.0	15.6	0
6 Conc C	0.12	0.33	1.84	2.05	20.0	20.25	0
1 Wood	0.20	0.47	0.77	2.10	15.25	15.25	7
2 Wood	0.19	0.35	0.87	2.30	17.0	17.0	7
3 Conc	0.06	0.20	1.29	2.13	18.5	19.6	7
4 Wood C	0.11	0.20	0.25	1.00	16.0	16.3	7
5 Wood C	0.06	0.11	0.42	1.92	17.5	17.5	7
6 Conc C	0.29	1.10	1.08	2.80	15.4	15.4	7

Note: C-Consolidated Ballast

*E.G. Cunney et.al., "The Effects of Accelerated Ballast Consolidation," Federal Railroad Administration, Report Number DOT/ORD-76/274, March 1977.

CHAPTER 5. TIE AND FASTENER COMPONENT PERFORMANCE

A major aspect of the assessment of track performance at FAST deals with the evaluation of individual components in terms of specified performance indices. These components include the rails, ties, fasteners, pads, insulators, ballast, and subgrade. This section is devoted to concrete tie/fastener and wood tie/fastener performance, while rail, ballast, and subgrade performance is discussed separately in chapters 6 and 7.

The performance indices utilized for the assessment of tie performance are the tie flexural strength under load, tie degradation and failure due to cracking, and excessive tie movement in track.

Fastener system performance is evaluated in terms of the fastener, pad and insulator movement and failure, gage retention, longitudinal rail movement and rail creep, and on wood tie fasteners, tie plate cutting.

A general description of sections 7 and 17 relevant to this discussion is given in appendix A. In section 5.1, concrete tie performance is looked at in terms of the performance indices stated above. More specifically, data from instrumented strain gaged ties yielding tie rail seat and tie center span bending moments were utilized to assess tie flexural strength under FAST loading conditions. Visual inspection data on concrete tie degradation, cracking failure, and tie movement data were analyzed for performance evaluation. In section 5.2, visual inspection data on fastener, pad, and insulator movement, and survey to benchmark data on gage and longitudinal rail movement were utilized for assessment of fastener system performance. In section 5.3, a short evaluation of tie plate cutting on different fastener/tie plate combinations on wood ties is presented.

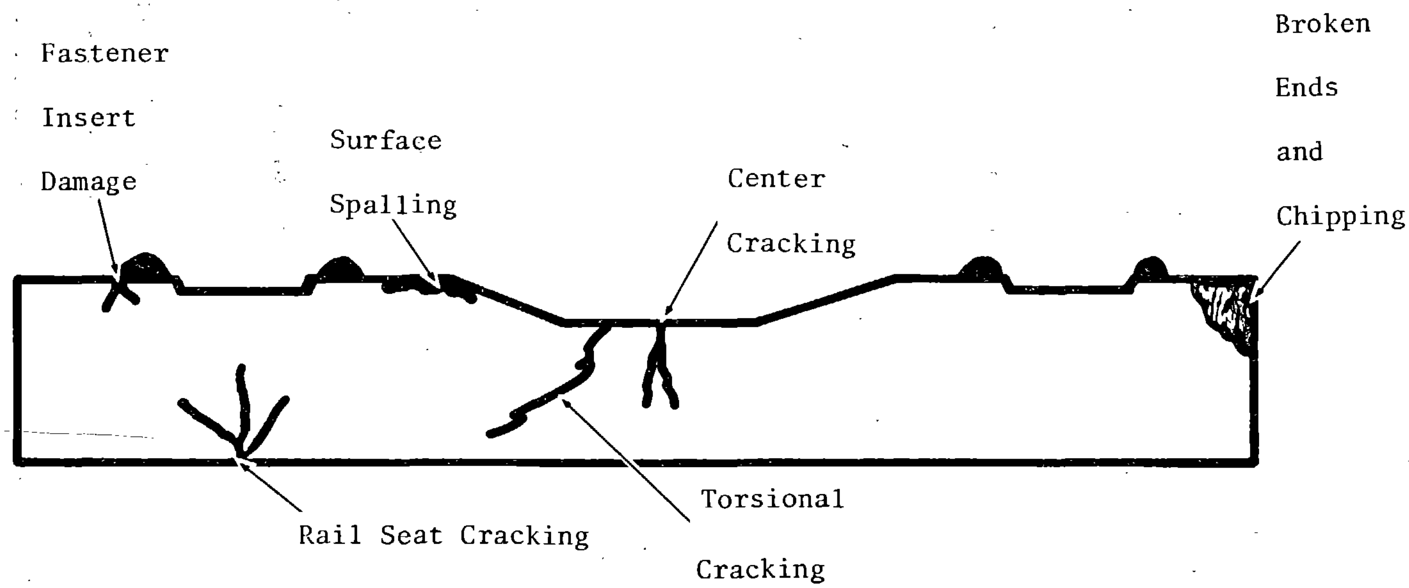
5.1 Tie Performance

The principal failure modes for concrete ties resulting from an overstressed condition are cracking in the rail seat area due to positive bending and cracking in the tie center due to negative bending and torsion (see figure 5-1). In terms of ballast support reactions, cracking in the rail seat area is usually attributed to the formation of voids under the rail seat caused by ballast flowing to the end of the tie rail seat or ballast degradation at the rail seat, both resulting in an "end-bound" tie. Tie center cracking due to negative bending is usually attributed to the increase in ballast support in the center region resulting in a "center-bound" tie.

Concrete tie failures may result from cumulative fatigue damage or an abrupt fracture caused by a single high load, such as from wheel flats. One critical loading parameter for concrete tie failures is the maximum bending moment in the tie.

The current industry specification for concrete ties is published in the American Railway Engineering Association (AREA) Bulletins 644 and 655[1] with some revisions given in Bulletins 650 and 660.[2] Flexural strength requirements include positive and negative maximum bending moments at the rail seat and tie center. These maximum required bending moments are specified for different tie spacings and tie length, based on static values for no cracking and dynamic tests within 30 days of casting. A repeated load test of 3×10^6 cycles of rail seat positive bending is also required for a precracked tie using a load range from 4 kips to 1.1 P, where P is the load required for the maximum static bending moment. It is expected that this high load represents the low percentage occurrence of high loads such as those due to locomotives and heavy cars and possibly wheel flats.

The first step in establishing concrete tie performance at FAST consists of evaluation of the load environment resulting from a consist made up of many



Note: Not to Scale

Figure 5-1. Tie Degradation and Failure Modes heavy cars, and determining the resulting maximum bending moments in the instrumented ties. The obtained bending moments are then compared with the specified values in AREA Bulletin 655 and test results from revenue service.

5.1.1 Tie Flexural Strength Under Load

Vertical wheel/rail loads were measured by means of rail web strain gage circuits at about 40 MGT at one location in section 17. The data were presented in a statistical format, as plots of probability density and probability distribution histograms. A typical probability density plot is shown in figure 5-2, where a clear demarkation between light and heavy wheel loads is evident. The mean peak load of about 33,000 pounds indicates a relatively high loading condition as compared to the mean loads of revenue service. This measurement is compared to existing vertical wheel/rail load data from the Union Pacific (UP), Florida East Coast, Southern Pacific (SP), and the Northeast Corridor (NEC) in terms of cumulative probability functions in figure 5-3. It can be seen that the large number of vertical wheel/rail loads at FAST are typically 30 to 50 percent higher than the revenue service environment. The question then is: What are the resulting maximum bending moments in the ties due to such severe loading conditions?

For the measurement of tie bending moment, strain gages were mounted on eight ties, four in tangent track (17E) and four in the 5° curve segment (17C). The gage locations and section properties of 17C and 17E are shown in figure 5-4 and table 5-1, respectively. Measurements were taken at 0, 5, 7 (following tamping), and 25 MGT. The strain gage readings were calibrated to yield bending moments, and the data is presented in terms of probability density and cumulative probability distribution histograms. A typical data sample is shown in figures 5-5 and 5-6. In addition to the distributions, the figures also contain, for each train pass, the speed of the train, the number of axles, the mean peak loads (bending moments) due to locomotives only, the mean peak loads (bending

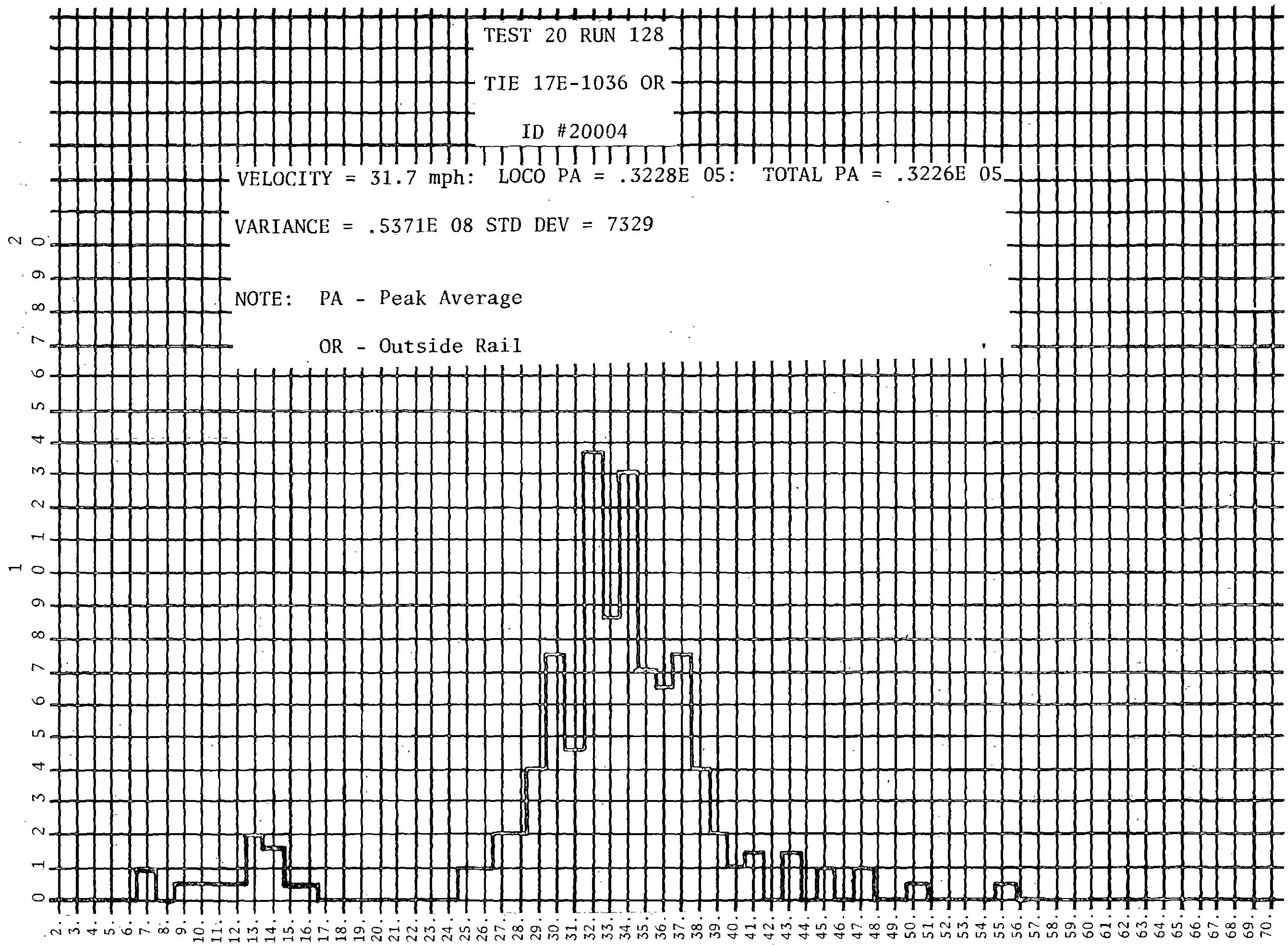


Figure 5-2. Typical Probability Density Plot of Wheel/Rail Loads

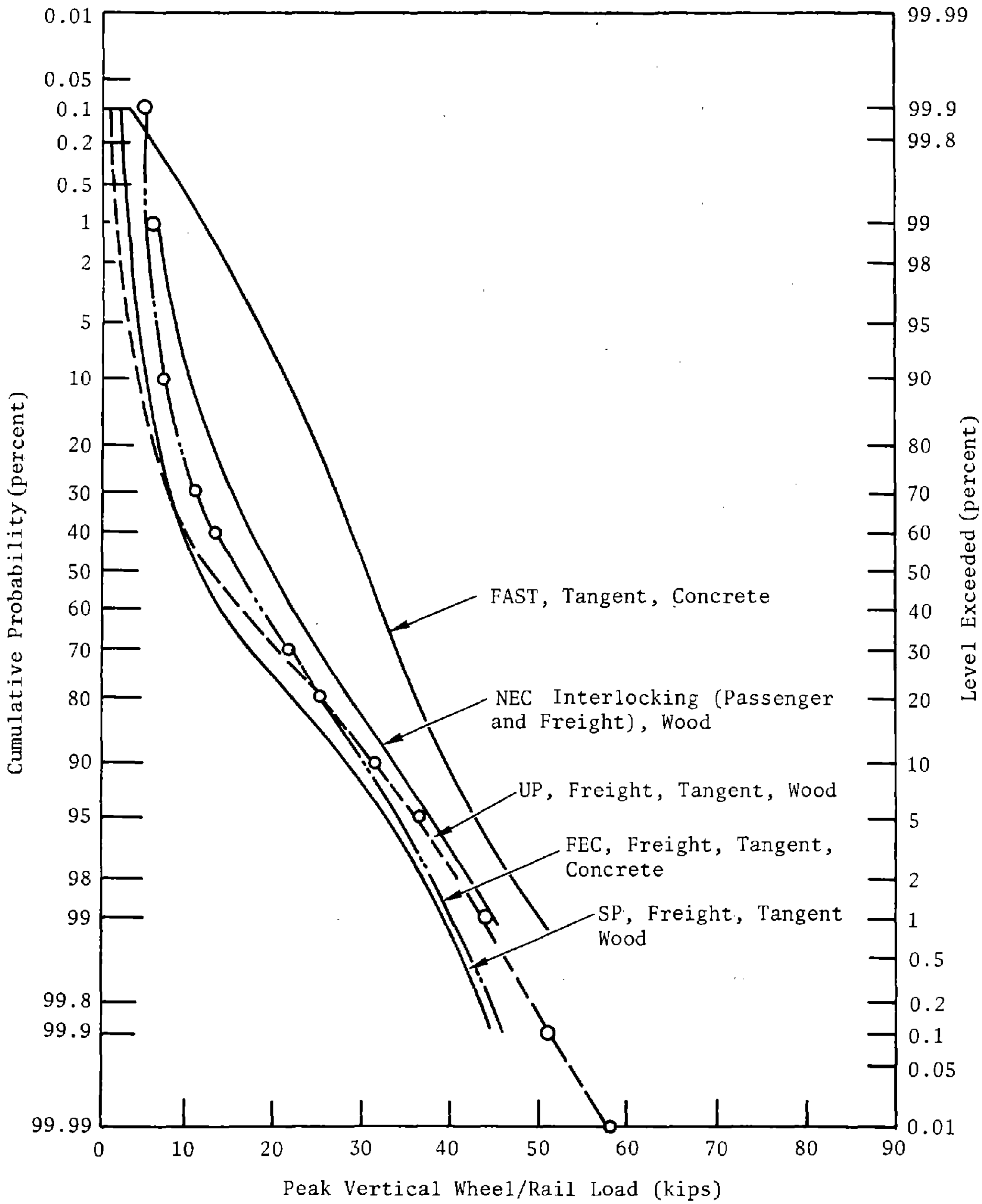
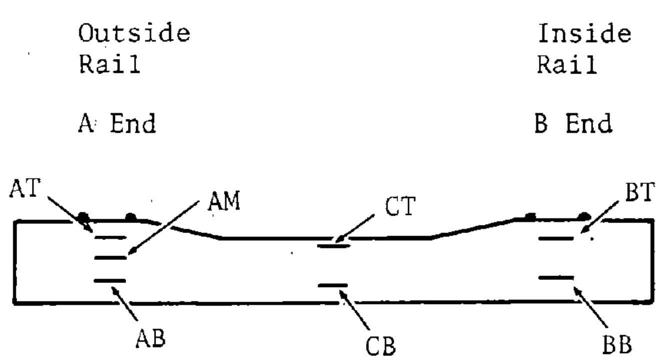


Figure 5-3. Peak Vertical Wheel/Rail Load Data Comparisons

Table 5-1. Concrete Tie Subsection Properties

Subsection Properties
17C; 17E
Length: 324 ft & 162 ties; 396 ft & 198 ties
Tie Type: T-1
Pad Type: P-2; P-3
Fastener: F-1
Ballast: 15" (Nominal)
Rail: Continuous Welded Rail, 136 lb (20 MGT @ Kansas Test Track)
Track: 5° Curve; Tangent
Tie Spacing: 24"



Not to Scale

Figure 5-4. Strain Gage Locations for Concrete Ties

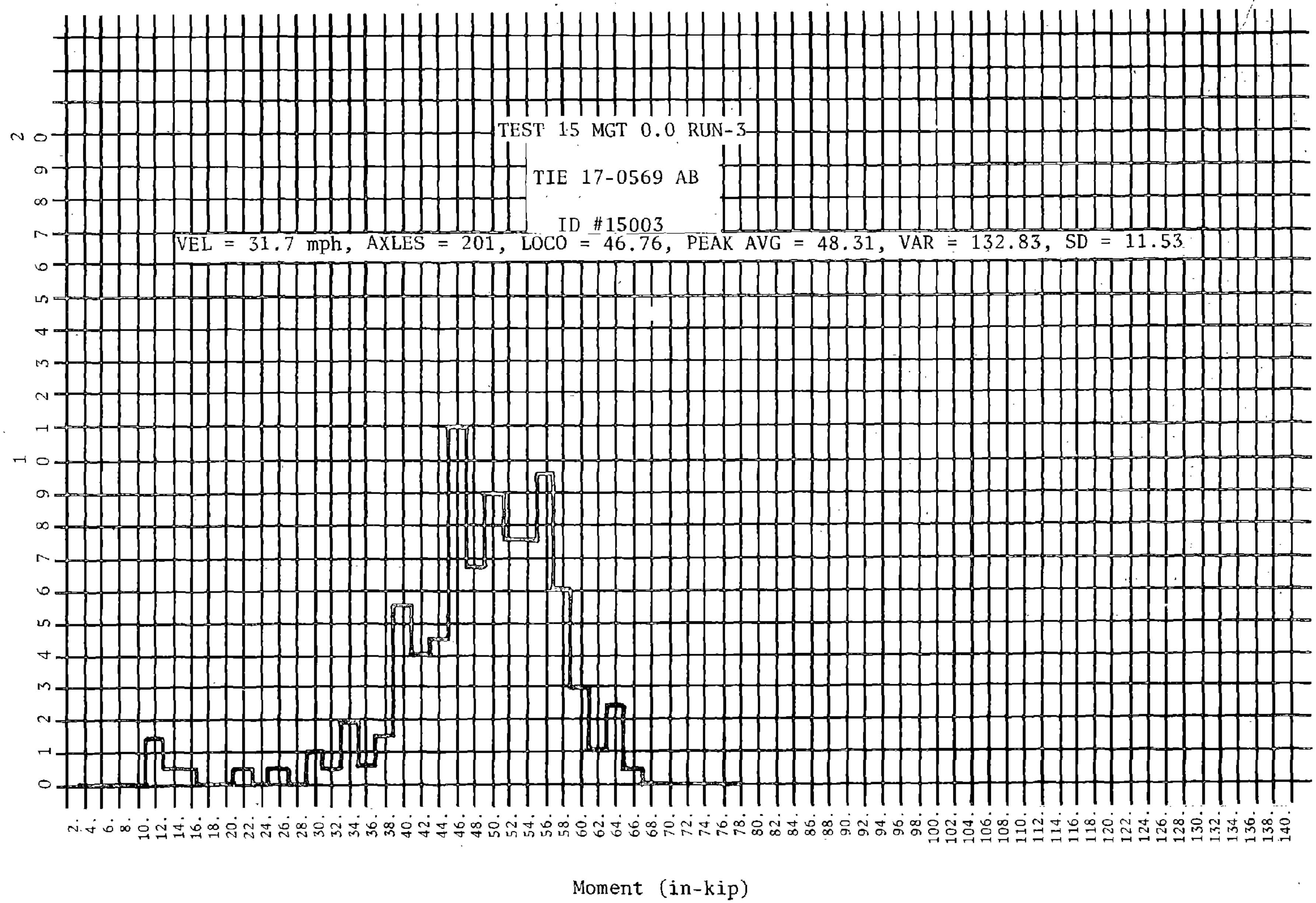


Figure 5-5. Sample Probability Density for Concrete Tie Bending Moment

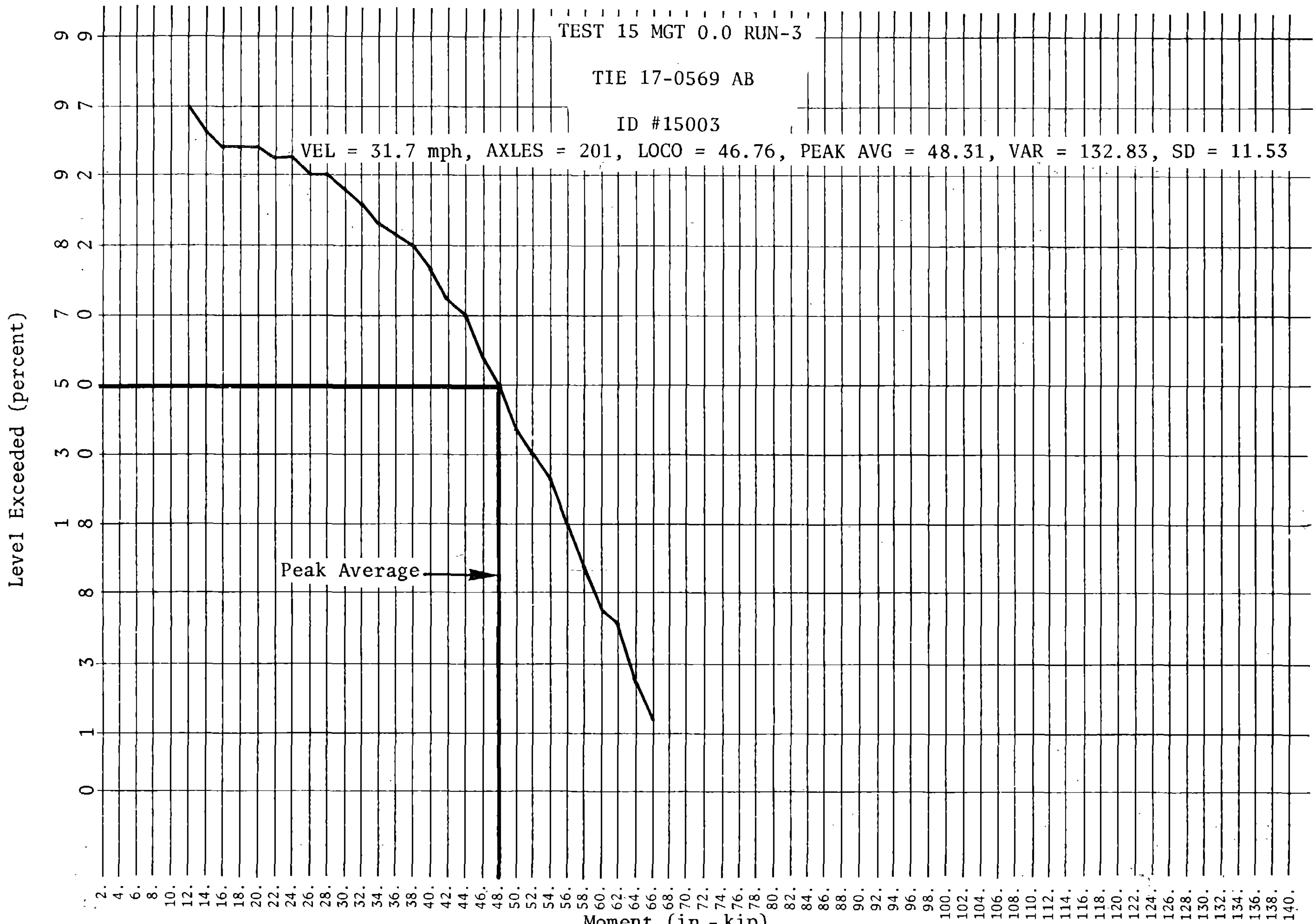


Figure 5-6. Sample Probability Distribution for Concrete Tie Bending Moment

moments) due to all cars, the variance, and the standard deviation. The average of the mean peak bending moments for the four ties, the average of the one-half percent occurrence (low probability) bending moments for the four ties, and the average of the mean peak bending moment due to locomotives only, were computed for each MGT level for both tangent and curved segments. These results are tabulated in tables 5-2 and 5-3.

The following can be noted from the results in tables 5-2 and 5-3:

a. The average rail seat bending moment decreases with tonnage typically in the order of 20 to 30 percent, while the midspan or center bending moment increases with tonnage in the order of 80 to 120 percent. This could indicate the early stages of a center binding condition.

b. The average peak bending moments due to locomotives are, in general, very close to the average peak bending moments due to all cars indicating that the loading effect of locomotives and heavy cars are almost identical at FAST.

c. There is no clear trend in bending moment variation from curved to tangent track.

The maximum bending moment measured at the rail seat was 80 in-kips. This was at 5 MGT in the tangent section. The maximum center bending moment measured was 130 in-kips, in the curved section also at 5 MGT. Concrete tie strain measurements of a somewhat similar nature were conducted in two other concrete tie test installations. One was at the Streator test track on the Atchison, Topeka and Santa Fe, where data was available from four strain gaged ties located in tangent track with two types of concrete ties.[3] The other test installation was on the Florida East Coast where concrete tie strain data are available for tangent and curved track and at different tie spacings.[4]

Table 5-2. Summary of Concrete Tie Bending Moment Statistics

Gage AB Rail Seat (High Rail)	CURVED			TANGENT		
	0 MGT	5 MGT	25 MGT	0 MGT	5 MGT	25 MGT
Average of Mean Bending Moment (in-kips)	37.8	N.A.	29.9	52.8	50.1	28.0*
Average of 1/2% Occurrence (in-kips)	54.7	N.A.	52.5	68.0	80.0	52.0*
Average Due to Locos (in-kips)	34.5	N.A.	34.5	53.5	48.6	24.9 *

*Based on only one tie measurement

GAGE CB (Midspan)	CURVED			TANGENT		
	0 MGT	5 MGT	25 MGT	0 MGT	5 MGT	25 MGT
Average of Mean Bending Moment (in-kips)	28.3	70.6	64.2	31.9	32.2	74.7
Average of 1/2% Occurrence (in-kips)	34.5	79.3	72.0	41.5	37.5	86.5
Average Due to Locos (in-kips)	24.5	71.8	65.2	31.6	32.9	78.2

Note: Locos - Locomotives only

Table 5-3. Summary of Concrete Tie Bending Moment Statistics

GAGE CT (Midspan)	CURVED			TANGENT		
	0 MGT	5 MGT	25 MGT	0 MGT	5 MGT	25 MGT
Average of Mean Bending Moment (in-kips)	48.7	69.0	71.8	31.8	29.3	72.3
Average of 1/2% Occurence (in-kips)	80.7	93.5	94.0	48.0	36.7	93.0
Average Due to Locos (in-kips)	39.4	73.0	75.1	31.8	29.6	76.5

GAGE BB Rail Seat (Low Rail)	CURVED			TANGENT		
	0 MGT	5 MGT	25 MGT	0 MGT	5 MGT	25 MGT
Average of Mean Bending Moment (in-kips)	37.0	22.3	24.3	36.0	43.7	27.1
Average of 1/2% Occurrence (in-kips)	57.0	38.0	44.0	49.5	59.3	45.3
Average Due to Locos (in-kips)	43.6	22.7	21.2	36.4	42.7	23.4

Note: Locos - Locomotives only

Table 5-4 shows a comparison of maximum rail seat and center bending moments measured at FAST, FEC, and Streator, and the requirements of the AREA Specifications. As can be seen, all measured data to date are considerably lower than those stipulated by the specifications. This seems to indicate that the ties have more than adequate flexural strength, and hence, no single overload flexural failures should occur. It must be noted that FAST data are from a limited number of ties and axle loads and probably does not include high impact loads that may exist at corrugations or temporary rail joints.

The requirements of the specifications and the low bending moment values measured in the various concrete tie test installations suggest an interesting question regarding the failure mechanism which governs concrete tie cracking. Is cracking failure induced by a single application of a very high load, or is failure caused by many cycles of much lower loads? It has been conjectured that small cracks are being initiated at load levels much lower than the static strength requirements of the AREA Specifications, and then once initiated, these cracks grow from the repeated loading of normal traffic.[5]

In summary, in spite of increased mean vertical wheel/rail loads, bending moments are lower than AREA requirements and compare well with results from other test tracks. All measured bending moment data is well within the strength requirements of the AREA Specifications.

5.1.2 Tie Degradation and Failure Due to Cracking

As shown in figure 5-1, the various tie degradation and failure modes are: rail seat cracking, center cracking, torsional cracking, surface spalling, fastener insert damage, and broken ends and chipping.

Visual inspection for these failure modes was conducted on 450 selected ties, from which 45 cribs were opened periodically to inspect 90 tie rail seat face areas for cracking.

Table 5-4. Comparison of AREA Flexural Requirements With Measured Data

	Maximum Rail Seat Bending Moment (in - kips)		Maximum Center Bending Moment (in - kips)	
	+	-	-	+
AREA*	250	115	200	90
FEC	78	-	55	67
Streator	97	-	72	-
FAST	80	-	130	-

*Based on 8'-6" tie length on 24" centers

Data from the last inspection (about 50 MGT) for rail seating cracking showed a total of 12 ties having minute cracks. Of these 12 ties, 10 were ties which were said to be cracked initially from being in service at the Kansas Test Track. The two other ties were one each off the T-3 and T-4 type, with one having a short (1 inch) crack at the rail seat and the other a long (11 inch) crack oriented 45^{o} from the bottom of the tie, outside the rail seat area and directed toward the rail.

Visual inspection of the 450 ties for top surface cracking and spalling, showed 20 ties having hairline cracks on the top surface near the tie end areas, and 3 ties with surface cracks in the center portion. Two other ties were removed from service due to being completely cracked through in the transverse (lengthwise) direction. One tie with a similar crack remains in service. Marks on these ties indicate that this damage was induced by a heavy blow to the fastener shoulder insert, probably in an attempt to drive in a loose clip or to realine the tie after skewing. Additional mechanical damage (such as due to tamping), broken ends and corners, and chipping were observed for eight ties. None of these flaws is considered the type to presently affect tie performance.

In summary, visual inspection data up to about 50 MGT show good tie performance in terms of tie degradation and cracking. It should be pointed out that all the cracks observed, except for the two ties removed, were classified as flexural or surface cracks as opposed to structural cracks. (Structural cracks at FAST were defined as cracks which open sufficiently to reduce the tie structural capacity or to create a loss of prestressing force, causing the tie to be unable to perform its required function.)

5.1.3 Tie Movement

Another major degradation mode for the track structure consists of excessive tie motion and movement in the ballast. This is important because it relates strongly to the performance of other components such as the ballast, the rail anchors, and the fasteners.

The latest visual inspection for tie movement conducted at 61 MGT showed 101 ties moving (only 3.5 percent of the total number of ties in section 17). Of these, 96 were in the 5° curve, 2 percent grade segment, out of which 23 exhibited longitudinal movement, with 9 ties moving more than 3 inches, and all 23 ties moving more than 2 inches. The remaining 73 ties exhibited varying degrees of skewing, with a tendency for greater tie movement on the inner rail side.

The same inspection revealed that in this 5° curve segment, 94 percent of the cribs around the skewed ties were half full, or less, indicating inadequate ballast maintenance or excessive ballast movement and flowing.

An additional concern in the same 5° curve segment was the initiation of considerable rail corrugations varying in wavelengths from 3 to 8 inches (5 to 6 inch average), and after 50 MGT, in amplitude from 0.04 to 0.07 inches, with more pronounced corrugations on the low rail. It is conjectured that the resulting high frequency vibrations induced into the ties by the wheel/rail interaction, results in ballast movement which produces a soft spot. As the track begins to settle at the soft spot, ballast crushing and tie skewing occurs, resulting in a progressive degradation of the track.

5.2 Fastener System Performance

Service history of rail fasteners used with concrete ties indicates that typical failure modes are: fracture, wear, and looseness of clips; deterioration and dislocation of pads; failure, separation, and movement of insulators; and pullout of fastener inserts. Adequate fastener performance is an important part of track performance as a whole, for it strongly affects gage retention, rail rollover, and longitudinal rail restraint.

At FAST, fastener system performance was monitored almost entirely via visual inspection for fastener, pad and insulator movement, and insulator failure.

5.2.1 Fastener Clip, Pad and Insulator Movement and Failure

Fastener clip movement relative to the tie shoulder was measured for 531 ties, for all 4 fasteners on a tie. Measurements up to about 35 MGT showed 42 fasteners with movement greater than 0.1 inch, with 80 percent of these fasteners located in the 5^{o} curve section, and mostly on the inner rail. Maintenance records indicate that, to date, 101 fasteners had to be reinstalled after coming loose, 70 percent of which were in the first five segments of section 17.

Tie pad movement was monitored for 622 ties for movement in lateral and longitudinal directions. Results indicate inner rail seat pad movement of 0.1 inch or greater for 60 pads and outer rail seat pad movement for 39 pads, the predominant pad movements being in the longitudinal (counterclockwise) direction.

Results of visual inspection data on fastener insulator movement for two sample sections (one curved and one tangent) are shown in table 5-5. As the data indicates, for the curved sample section, approximately 79 percent of the insulators moved on the inner rail (both field and gage sides), with 75 percent of the insulators moving by at least 0.1 inch in the longitudinal direction down the grade. On the tangent sample section, all insulator movement was in the same longitudinal direction, with 40 percent moving on the inner rail field side versus 24 percent on the inner rail gage side. The absolute magnitudes of the insulator displacements were much larger for the curved section than for the tangent segment.

Maintenance records to date indicate that the total number of insulators replaced is 40 out of about 10,480.

Table 5-5. Tie Insulator Movement

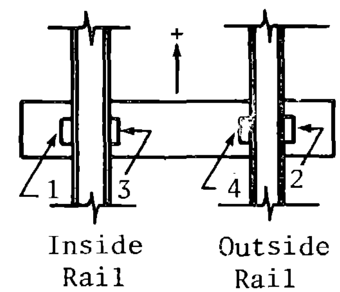
Insulator Number	% MOVING IN + DIRECTION	% MOVING IN - DIRECTION	% MOVING $\geq \lvert 1.5'' \rvert$	% MOVING $\geq \lvert 1.0'' \rvert$	% MOVING $\geq \lvert 0.5'' \rvert$
1	4	75	3	8	40
2	27	10	7	2	5
3	4	77	5	11	42
4	17	15	1	2	3

Sample:

Ties 17-0350-0375

17-0460-0487

5° Curved Track



Insulator Number	% MOVING IN + DIRECTION	% MOVING IN - DIRECTION	% MOVING $\geq \lvert 1.5'' \rvert$	% MOVING $\geq \lvert 1.0'' \rvert$	% MOVING $\geq \lvert 0.5'' \rvert$
1	0	40	0	0	4
2	0	18	0	0	0
3	0	24	0	0	1
4	0	8	0	0	0

Sample:

Ties 17-1331-1380

Tangent Track

Note: Data at 30 MGT

Minimum Measured Movement = 0.1 inch

A "quick look" concrete tie/fastener performance comparison with the test installation at Lorraine on the Chessie System is shown in table 5-6. The Chessie test section consists of 224 concrete ties (118 RT7-S and 106 Costain CC244) with 24-inch tie spacing on granite ballast on a 3°curve. The fasteners are the Pandrol 601 and the CS-5 types. The data presented in table 5-6 is at approximately the same MGT level of traffic, and the comparison indicates about the same tie and insulator performance at FAST as at the Lorraine test section. However, FAST pad movement performance is superior to the Chessie results.

5.2.2 Gage Retention, Longitudinal Rail Movement, and Rail Creep

One major advantage of the concrete tie/fastener over wood tie/tie plate/ cut spike construction is the improved restraint characteristics in terms of lateral and longitudinal rail movement resulting in improved gage retention and reduced rail creep. Consequently, in evaluating fastener system performance for concrete ties, gage retention longitudinal rail movement or rail creep are important parameters.

Static gage measurements at 100-feet survey to benchmark locations throughout the entire FAST loop were used to compare gage variation with MGT for concrete versus wood tie track. A sample comparison of mean gage versus MGT for concrete versus wood tie track (both on 5° curve) is shown in figure 5-7. It can be seen that in this sample of 5° curve data the gage is similar for both tie types. Note that there is excellent gage retention for tangent track on concrete ties. It should be pointed out, however, that based on a preliminary analysis of rail head wear data, rail wear at the gage point contributes a major portion of the increasing gage on curves for both wood and concrete ties. Gage data in figure 5-7 includes this contribution. Since the wear data (see 6.3) is similar for both curves, it is evident that at FAST, at this level of MGT, the gage retention capabilities for wood and concrete ties from data taken statically, are similar.

Table 5-6. Tie and Fastener Performance Comparison

	CRACKED TIES	FAILED INSULATORS	DISLOCATED PADS
CHESSIE (LORRAINE)	5%	0.5%	34%
FAST	1%	0.4%	8%

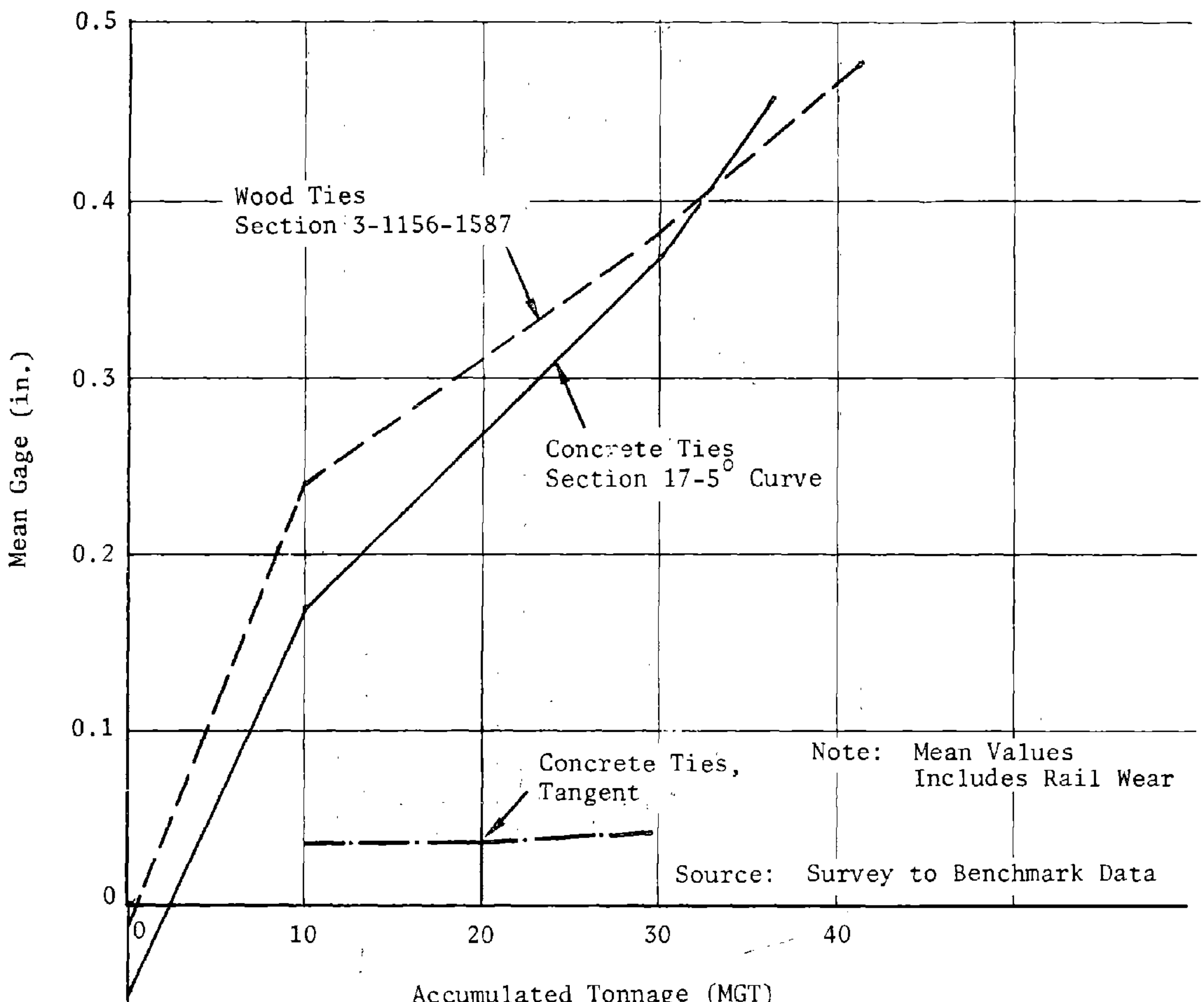


Figure 5-7. Mean Gage vs. Tonnage

However, other test results indicate better gage retention on sharp curves for concrete versus wood tie track.[6] Creep and longitudinal rail movement measurements indicate better performance for concrete ties than for wood ties, except in the area of excessive tie motion (such as the soft spot on the 5° curve -- see section 5.1.3).

5.2.3 Rail Fasteners on Wood Ties

The test section for the assessment of rail fasteners on wood ties is described in appendix A. Due to the shortness of section 7, only the tie plate cutting versus tonnage is covered in this report.

The data in table 5-7 on tie plate cutting were taken just prior to the rail being transposed in section 7 on 33.1 MGT and compared to a zero MGT baseline. In each of the five fastener groups, there was a minimum of six measurement locations on both the high and the low rail. The data shows that there was a negligible amount of tie plate cutting on the low rail. However, on the high rail, field side edge of the plate, there was some tie plate cutting, with four of the five fastener types. There was no tie plate cutting with the elastic spring type fastener.

Even though the mean of the amount of cutting was small, varying from 0.025 to 0.045 inches, it was significantly different at a 97.5-percent level of confidence from the cut spike at the rail segments versus the elastic clip. The data on the compression clip showed too much scatter to draw any conclusions.

5.3 Related Experience

In addition to FAST, there is performance data available from other concrete tie track installations. In this country, test installations at Streator on the Atchison, Topeka and Santa Fe, at Lorraine on the Chessie System, and at Roanoke on the Norfolk and Western, all contain concrete ties meeting the AREA Specifications. To date, these test installations exhibit good performance in terms

Table 5-7. Tie Plate Cutting in Section 7

Fastener Type	Tie Plate Type & Size	Mean Vertical Tie Plate Cutting (in.)
2 cut spikes/rail 2 cut spikes/plate	1:40 7-3/4" x 14"	0.045
2 cut spikes/rail 2 lock spikes/plate	1:40 7-3/4" x 14"	0.025
2 compression clips/rail 2 cut spikes/plate	1:40 7-3/4" x 14"	0.025
2 elastic clips/rail 2 lock spikes/plate	1:40 7-3/4" x 15"	0.00
2 cut spikes/rail 2 screw spikes/plate	1:40 7-3/4" x 14"	0.037

Note: All Data for High Rail

of retaining gage, line, and surface, and acceptable performance of the fastener system.

The Canadian National Railways are sufficiently convinced by results of a previous 10,000 concrete tie test installation, in terms of extended rail and tie life and reduced maintenance projections, to embark on a major concrete tie installation program on territories where extreme curvature exists. They are purchasing 1.5 million ties to be installed over a 5-year period.[6]

Foreign experience such as from the Soviet Union, French SNCF, the Australian Railroad, and the German DB also indicates favorable performance in terms of increased tie life and less maintenance. Additional data on concrete tie experience is summarized in table 5-8.[7]

The elastic fasteners on wood appear to reduce tie plate cutting. However, due to the short traffic duration and FAST test section, no substantive data on the fasteners ability to retain gage and to restrain the rail longitudinally were available. These evaluations must be made before any definitive conclusions as to the fastener performance can be made. Several field installations are in revenue service; however, there are no conclusive results to date.

Table 5-8. Concrete Tie Experience

LOCATION	COMMENTS
Lorraine & Streator	Two types of ties on curve and tangent; 15 - 18 MGT annual; good line and surface, no ballast degradation; minimal fastener movement, some pad and insulator movement; no major cracks or failure
Canadian National Jasper, Alberta	CN installing ties on curves up to 8^{o} on existing embankment; increased rail life; out of 10,000 ties 11 had been removed in 4 years service, 600 ties exhibit minute surface cracks (0.3% of which were structural); surfaced once in 4 years; had not been lined; comparable wood tie track would have required more surfacing and lining; 40 MGT annual
Chessie (Noble)	Gives relative track performance for new installation of concrete and wood ties; wood ties spaced at 23" and concrete at 25" and 27"; track modulus was 2,500 vs 13,000 psi; during initial 15 MGT service wood tie track settled 1-1/2" to 2" more than concrete tie track; over 5 years track settlement for wood ties was twice that of concrete tie track
Hamersley Iron Railway in Australia	Ties at 24 - 27 inch spacing; some problems with insulators and fasteners; some centerbinding cracks; the most important benefit is the decreased level of maintenance compared to standard wood tie track while maintaining a high standard of surface and line
French Nat'l RR (SNCF)	relatively good results; life about twice that of wood ties; much less maintenance; less satisfactory retention of track surface on poor roadbed
Soviet Railway	25% of their mainline is concrete tie track; dual pad system, 1/4" thick; entire track panels are removed after an average of 550 MGT to inspect rail and replace rail and any defective ties: 12% are replaced after 550 MGT; maximum axle loads are about 24 tons.

CHAPTER 6. RAIL WEAR COMPARISONS

The five principal types of rail metallurgy and heat treatment, in sections 3, 7, and 17 on the 5° curve, were analyzed for wear patterns. The five types are standard control cooled carbon, high silicon, fully heat-treated, head hardened, and chrome molybdenum. The objective was to determine the differences in gage point wear among the different rail metallurgies, heat treatments, and locations. Because of the unusual rail wear pattern in evidence at FAST, a result of the unit train operation and the near uniform flange height on the wheels, a methodology was developed for determining rail wear. Instead of computing the rail head area loss, the rail head contour was measured at six points. Since the analysis was primarily concerned with the wear on the high rail, wear at the gage point 5/8-inch from the top of the rail was analyzed. The method of locating that point always used the original fixed datum so that the measurement location was always in the same place on the rail.

6.1 Rail Metallurgy

In section 3, a comparison of gage point wear among the different rail metallurgies was made on the 1:40 cant tie plate (see figure 6-1). The data, which were averaged from throughout the section, showed that there were, based on statistical evaluations, three wear groups: (standard), (fully heat-treated and high silicon), and (chrome molybdenum and head hardened). These trends are consistent throughout the section. However, due to data scatter and to different wear rates in the curve any further categorization was not statistically valid. The data were from profiles taken up to 40 MGT.

6.2 Tie Plate Cant

When comparing the rail wear on the different tie plate cants (see figure 6-2) the data showed a trend which indicated that the gage point of the rail was wearing greater on the 1:14 cant tie plate than the 1:30 and 1:40 cant tie

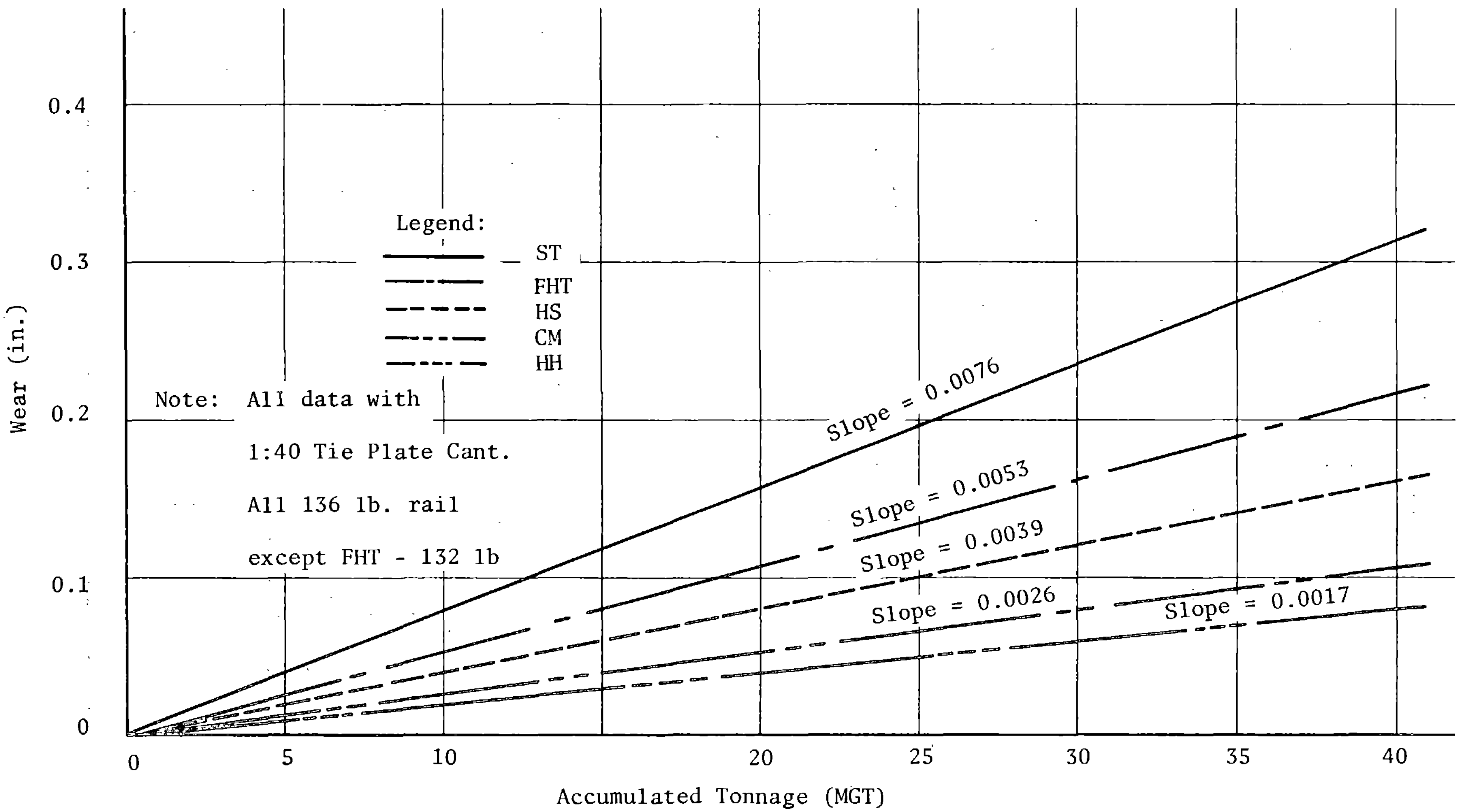


Figure 6-1. Gage Point Wear vs. Tonnage

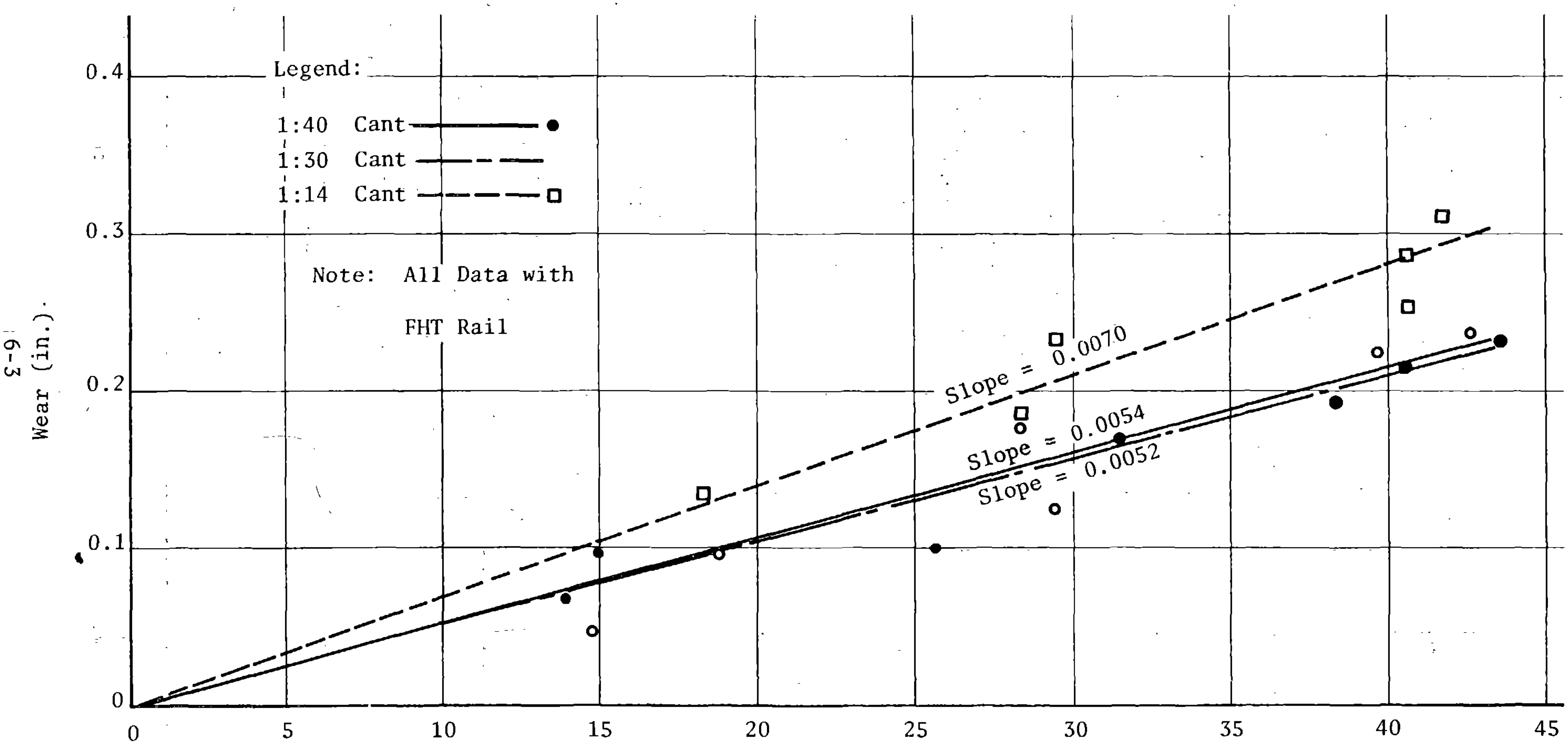


Figure 6-2. Gage Point Wear vs. Tie Plate Cant

plates. Even though the trend was evident for each of the rail metallurgies in section 3, the differences in rail wear among the test segments with different tie plate cants were not considered statistically significant.

6.3 Standard Rail

A comparison of gage point wear on the 5^{o} curves in sections 3, 7, and 17, all on a 1:40 cant tie plate was made (see figure 6-3). It demonstrated that there was a significant difference on the wear between section 7 and sections 3 and 17. This largely was due to the nonuniform rail lubrication around the FAST loop during the first 50 MGT and consequently should not be attributed to the variations in the track structure. The trend in sections 3 and 17 shows that there is slightly less rail wear on concrete ties than wood ties. However, the results of the data analysis are not considered to be statistically significant.

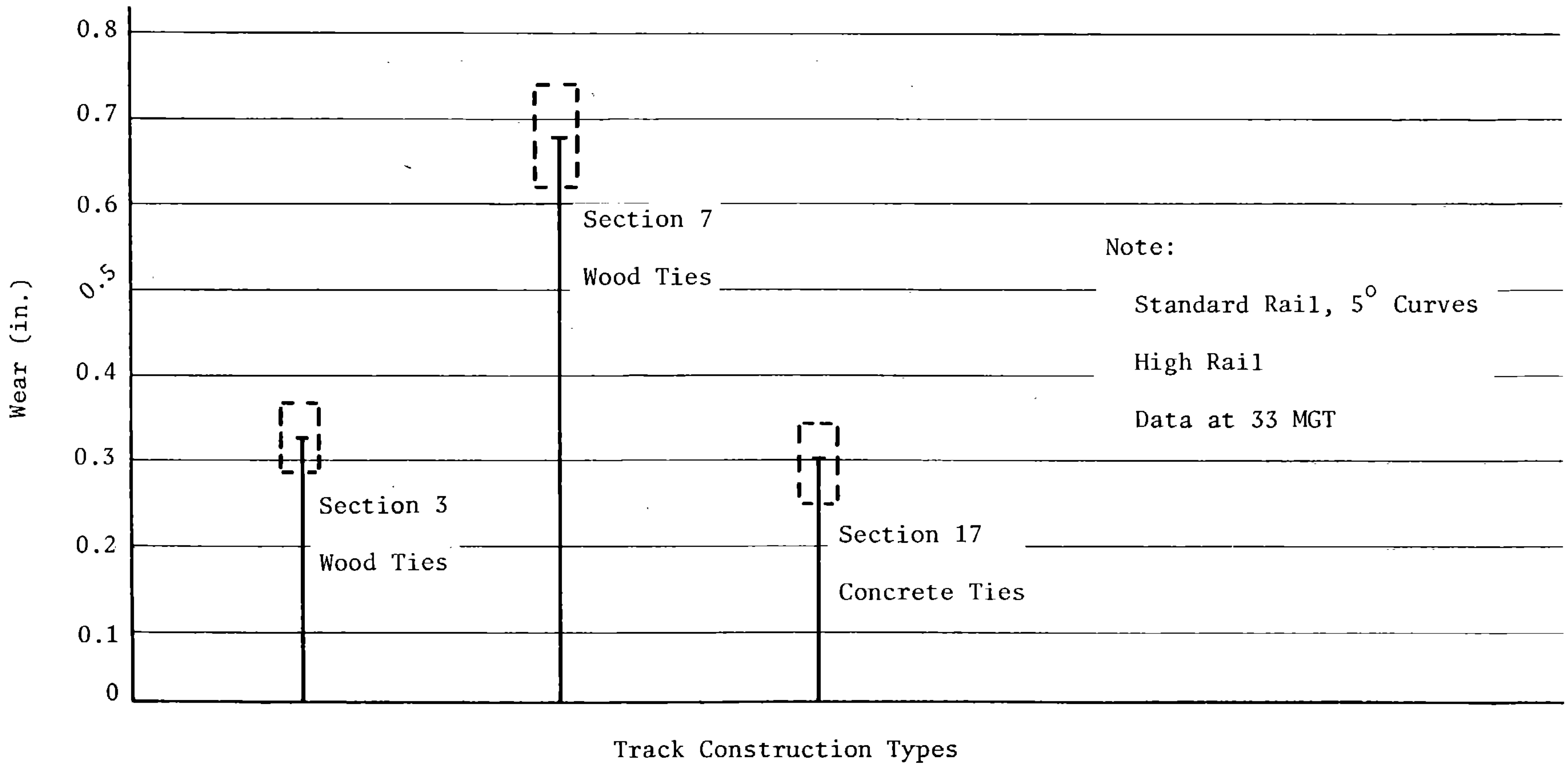


Figure 6-3. Gage Point Wear - Standard Rail

CHAPTER 7. BALLAST AND SUBGRADE PERFORMANCE

This chapter relates the performance of the FAST track substructure (ballast, subballast, and subgrade) to variations in the tie, fastener, and ballast type; ballast depth; and ballast shoulder width as monitored by the many FAST measurements and observations. Specifically, it will present the analyses that have been made on FAST data as they relate to overall track and track substructure component performance.

7.1 Tie Type Comparison

The loading conditions imposed by concrete tie track are different from those of wood tie track and the variation in substructural response to these conditions can impact track performance. The effect of tie type on overall track performance has been discussed in chapter 4. In this section, the effect of tie type on substructural component performance is analyzed. The specific measures of performance to be utilized are vertical pressures on subgrade, subgrade deformations, ballast deformations, and ballast gradations.

In all instances, the comparison of tie type is made with other variables held relatively constant. The one major exception that is bound to influence relative performance and be difficult to isolate in the analyses is rail type. The concrete tie track has continuously welded rail (CWR) while comparative wood tie sections have jointed rail. The two sections of greatest interest for performance comparison are sections 17E and 18B. Both are tangent track with granite ballast 14 to 16 inches deep. Each section is new construction with 17E having concrete ties and 18B wood ties.

7.1.1 Vertical Pressures

Soil stress cells were used to measure vertical pressures under the tie rail seat at the subballast/subgrade interface. Four 10-inch square inductance coil soil stress cells in each section (17E and 18B) were placed after track construction but prior to the commencement of train operations. Uniform placement procedures were used, but the depth of the ballast/subballast zone in 17E (18 to 19 inches) was slightly different from the wood section, 18B (20 inches). Concrete ties were beneath CWR and at 24-inch tie centers while wood ties were on 19½-inch centers under jointed rail; however, these test sections were located away from joints. Otherwise, the test sections were identical.

Preliminary analyses of vertical pressure directly under the loading point of the ties, at the top of subgrade after approximately 40 MGT of traffic showed virtually identical results for concrete and wood tie track. The mean measured pressure under the ties was 5 ±1 psi, indicative of almost identical vertical load conditions at the measured point. The magnitude of vertical stress appears to be reasonable, but further data analysis is necessary to increase the precision of the numbers and verify the magnitude and load path.

7.1.2 Subgrade Deformations

The subgrade deformations were measured with inductance coil extensometers anchored up to 10 feet below the top of subgrade. Tops of the extensometers were located below plates placed at the subballast/subgrade interface. The extensometers had to be placed approximately 6 inches inside the rail to accommodate the augers used in constructing the boreholes into which the extensometers were placed.

Measurements (figure 7-1) indicate that after approximately 1.5 MGT of traffic the subgrade under the concrete tie track began deforming at a greater rate than the subgrade under the wood tie track. The curve (deformation with accumulated load) appears to indicate that although the rate of deformation under wood tie track was beginning to decrease, the same was not true under the concrete tie track. Observations reported by Selig and Adegote[1] at the time of extensometer installation indicated that the subgrade appeared to have uniform

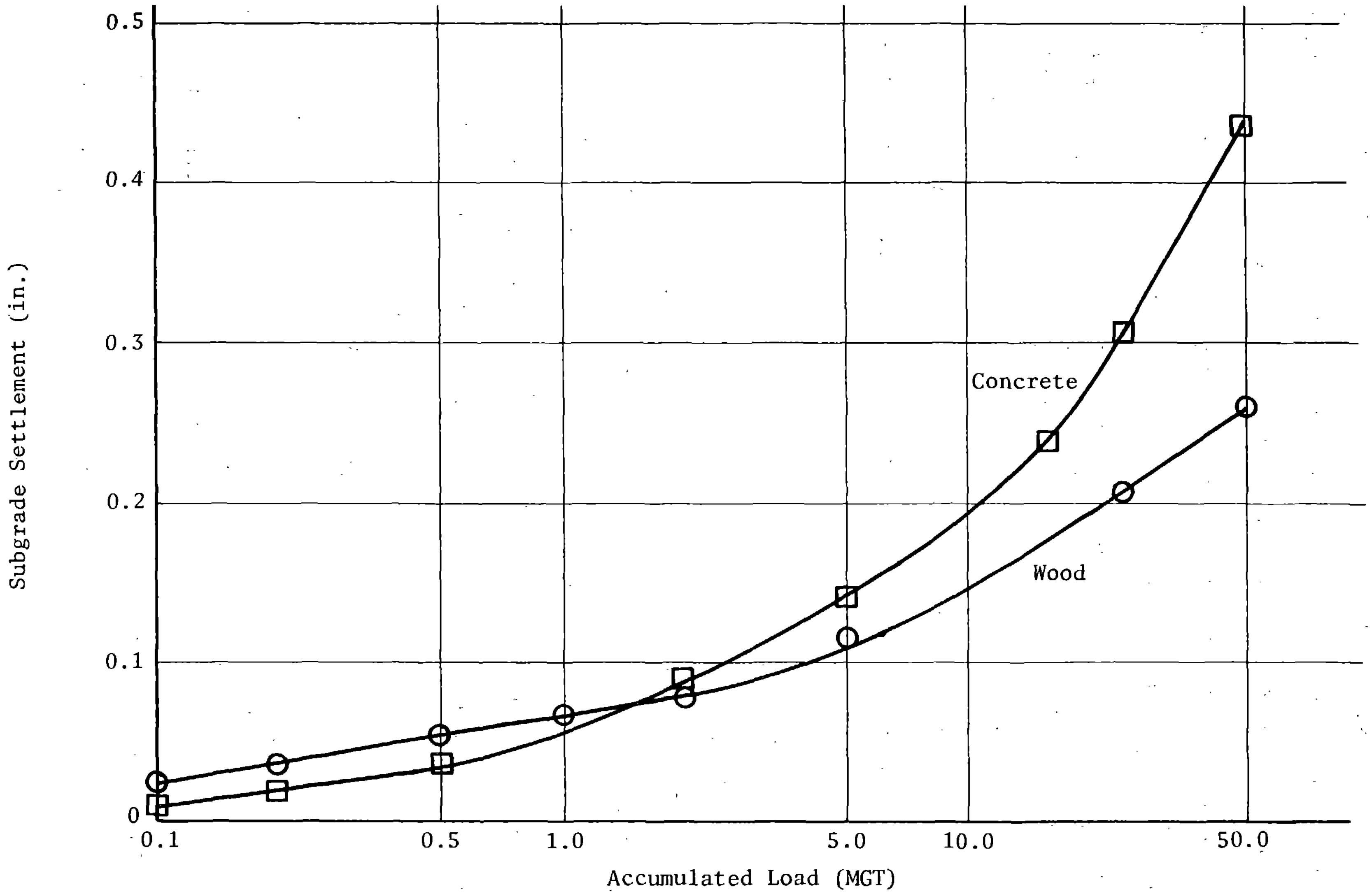


Figure 7-1. Subgrade Deformation with Load - Concrete vs. Wood Tie Track

properties throughout sections 17, 18, and 20; but no actual measurement of subgrade deformation properties was made at that time. Water contents measured in the subgrades in sections 17 and 18 were approximately the same.

For the relatively good embankment material at the FAST track, the trends shown in the concrete tie sections, if continued, would begin to become a source of concern with respect to potential impact on track performance. The instrumented sections, because instruments were installed after track construction, are potential relative "soft" spots and would be expected to settle at a greater rate than undisturbed track. However, the differential in mean subgrade deformation under concrete and wood tie track may be indicative of a general trend. This trend certainly contradicts the measured pressures mentioned earlier if one assumes that the measured pressures are accurate and that vertical pressure is the controlling parameter in the subgrade deformation mechanism.

The much higher stiffness of the concrete track may be a possible cause of the difference in deformation rates. Although of equal magnitude at point of load, the higher concrete tie track stiffness and wider tie spacing may lead to a much less uniform distribution of vertical pressures over the subgrade which may be causing the concrete ties to "punch" into the subgrade. It is possible that, after some amount of load has been accumulated on the track, stress distributions will have shifted sufficiently to slow or stop this deformation.

7.1.3 Ballast Deformation

Measurements of ballast deformation were made using inductance coils at a vertical spacing of approximately 10 to 14 inches on center. In wood tie track, the top coil was countersunk into the tie, while in concrete tie track it was taped to the bottom of the tie. Unfortunately, the coils of most interest, those in the top section of ballast, were very sensitive to any maintenance performed in the test section. This was especially significant in the concrete tie section which was surfaced at approximately 7 MGT because of initial track alinement irregularities. Deformation data, however, were reconstructed to present smooth curves representing the deformation behavior of a virgin (untouched) ballast section (see figure 7-2).

It is evident from figure 7-2 that, in the first 50 MGT of track life, the ballast in the concrete tie section was settling a greater amount (both initially and also at a greater rate) than the ballast in the wood tie section. This observation is consistent with observations at other facilities, as mentioned in chapter 4, and is also consistent with current compaction theories. The heavier concrete track provides more confinement (lateral stress) to the ballast, and therefore consolidation of the ballast occurs at a faster rate. Comparing the field observations of displacement with established behavior of granular material under cyclic load, it would be anticipated that larger, earlier settlement would occur in concrete tie track with initially larger differential settlements. However, as the track continues to accumulate traffic beyond 50 MGT, it would be expected that the rate of settlement in concrete track would begin to decrease, and the total settlement under the wood tie track would begin to approach that of the concrete tie track. The implication on track performance of the relative difference in deformation behavior is that it would be anticipated that much more settlement in concrete tie track would occur initially after construction and maintenance, attempting to minimize differential deformations initially, or the use of a ballast consolidator, could be very beneficial to concrete tie track performance.

7.1.4 Ballast Gradation

The ballast in sections 17, 18, and 20A was granite, all from the same source. The material was all finer than AREA number 4 Specifications (see figure 7-3a) which was inconsistent with ballast standards for the remainder of the FAST loop. This inconsistency in standards probably had little influence

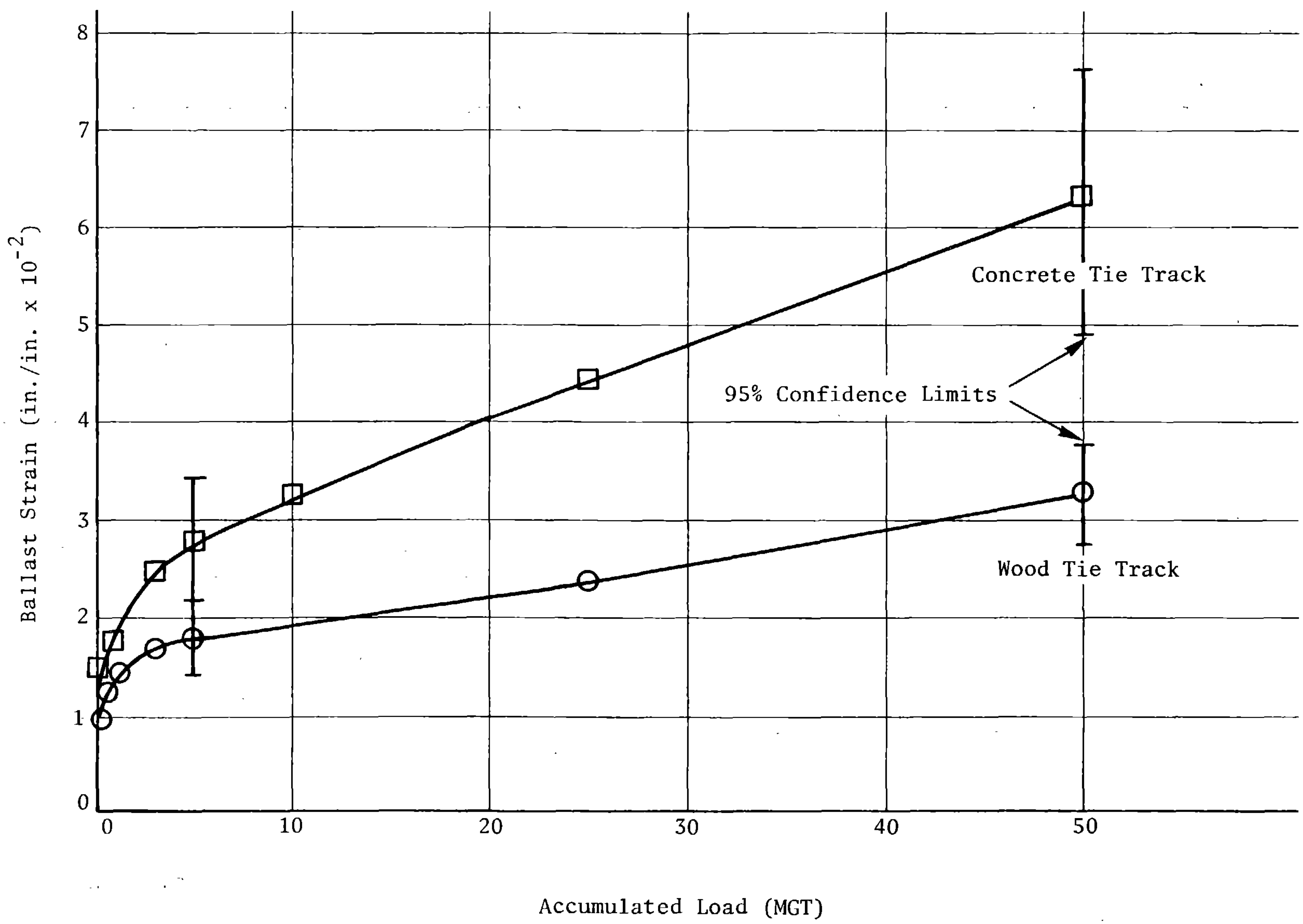


Figure 7-2. Ballast Strain with Accumulated Load - Concrete vs. Wood Tie Track

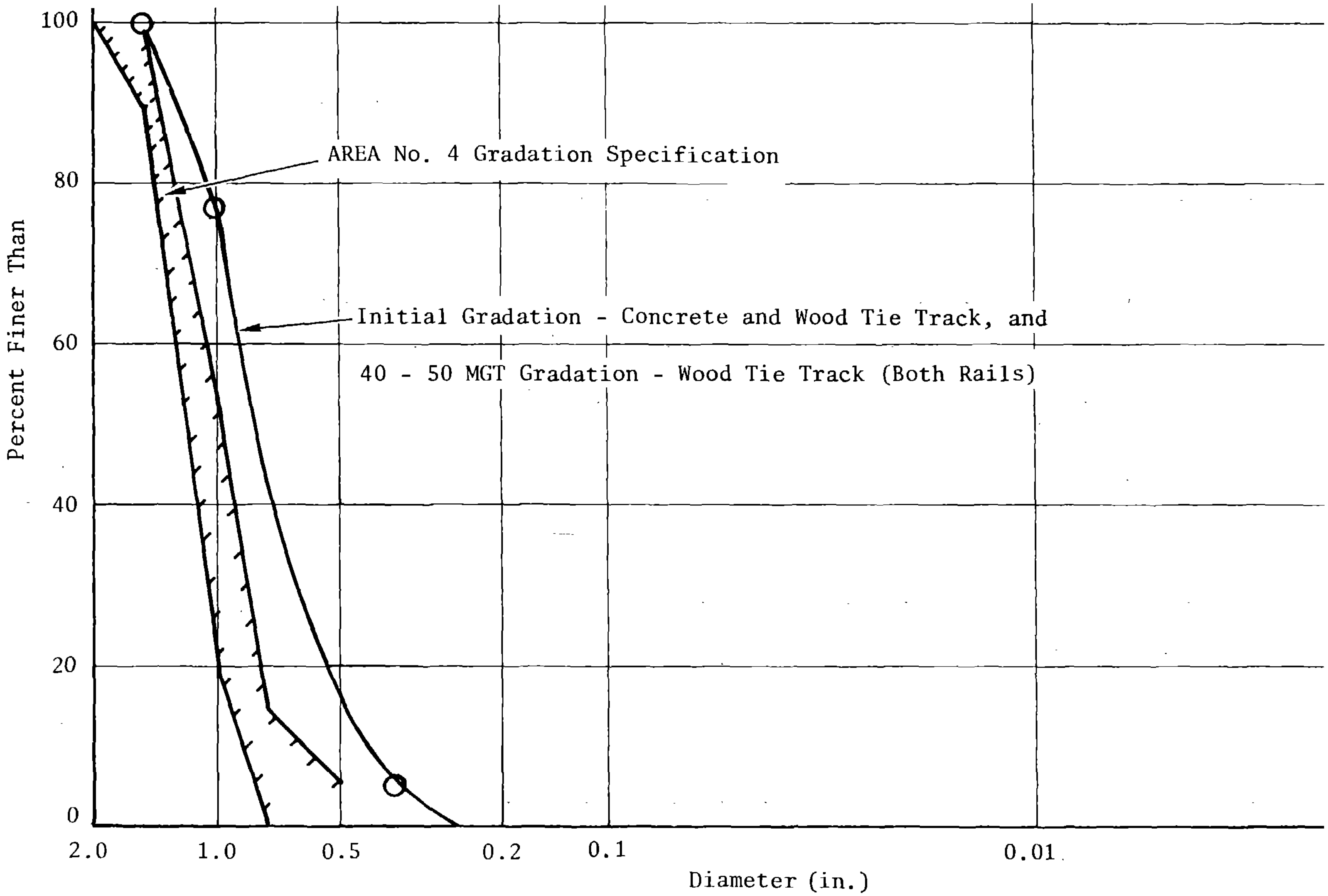


Figure 7-3a. FAST Granite Ballast Degradations

on the performance of concrete or wood tie track with respect to ballast degradation, since the gradation for the comparison sections was the same.

Ballast samples were collected from under wood and concrete ties, and a sieve analysis run to see if any effect of tie type on ballast degradation could be observed. As shown in figures 7-3a and 7-3b, no changes in gradation could be observed under four wood ties and three concrete ties, all on tangent track. Two of the ties pulled for samples on the wood tie track were in so-called "soft" spots where the tie was observed to be in a slight center-bound condition.

Comparing these results with the gradation of samples taken from two ties in a maintenance trouble spot on curved concrete tie track (figure 7-3c), it can be seen that rather significant ballast particle degradation has occurred under the outer (or high) rail of the concrete tie track.[2] These degradations were observed at a maintenance trouble spot on concrete tie track (a location with a paucity of in-crib ballast, and observed rail corrugations and tie skewing), but not on wood tie track "soft" spots. This may indicate that "soft" spots are a more serious hazard and are precursor to further maintenance difficulties on concrete tie track than on wood tie track.

It should be recalled that the observations made above apply for up to approximately 50 MGT of a rather severely loaded track resting on a displacement resistant foundation consisting of a cementing sand. Significant trends may just be beginning to develop, further observations, and possibly some refinement of the measurements being taken could shed substantial additional light on the questions posed. Trends observed above may not apply at all to other conditions of foundation support, and care especially should be taken if trying to relate the above data to clay subgrades.

7.2 Ballast Depth and Type Comparison

Two major experiments involve a study of the effect of variations in ballast type and depth on overall track performance. The two sections used primarily

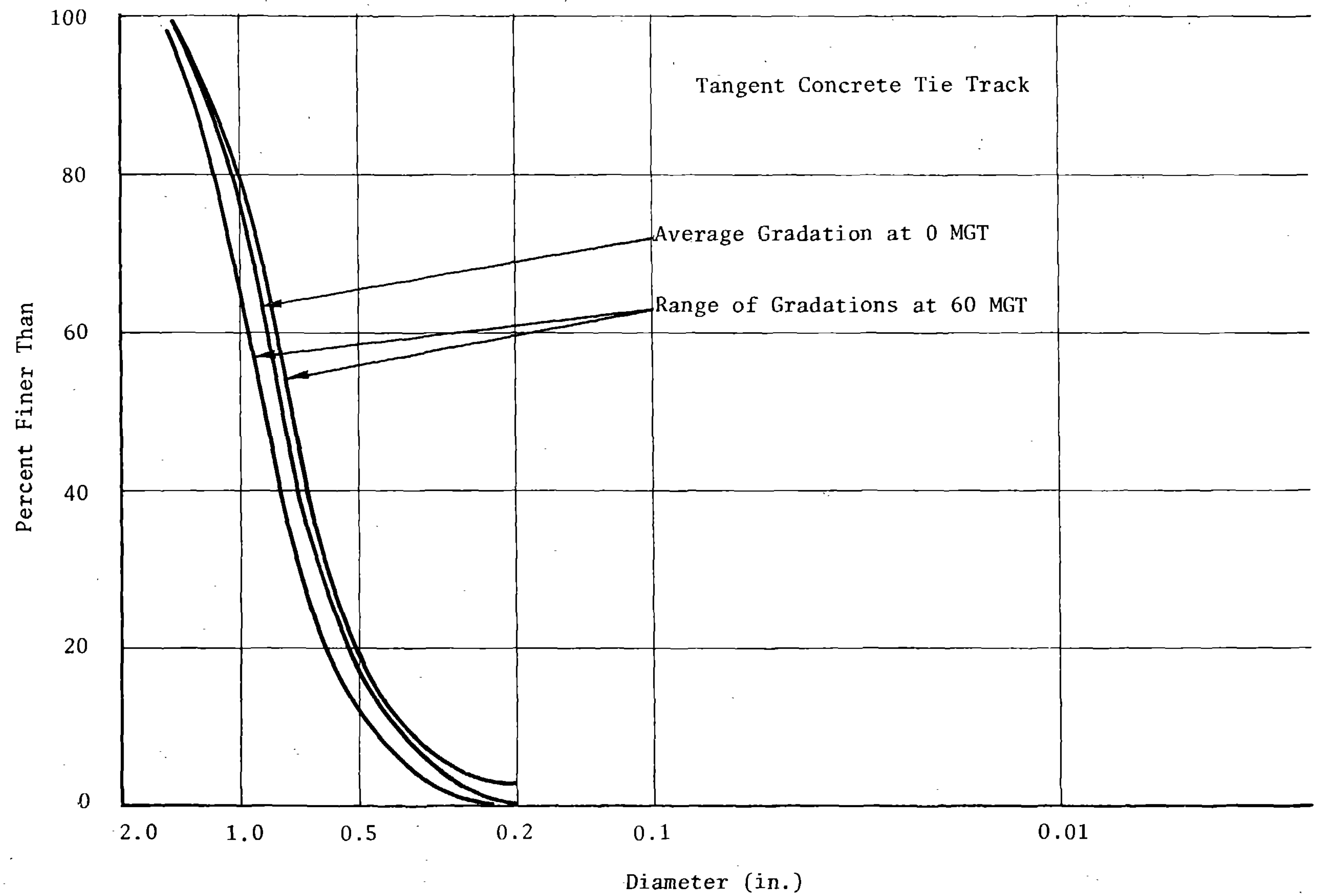


Figure 7-3b. Ballast Gradation Under Concrete Tie Track

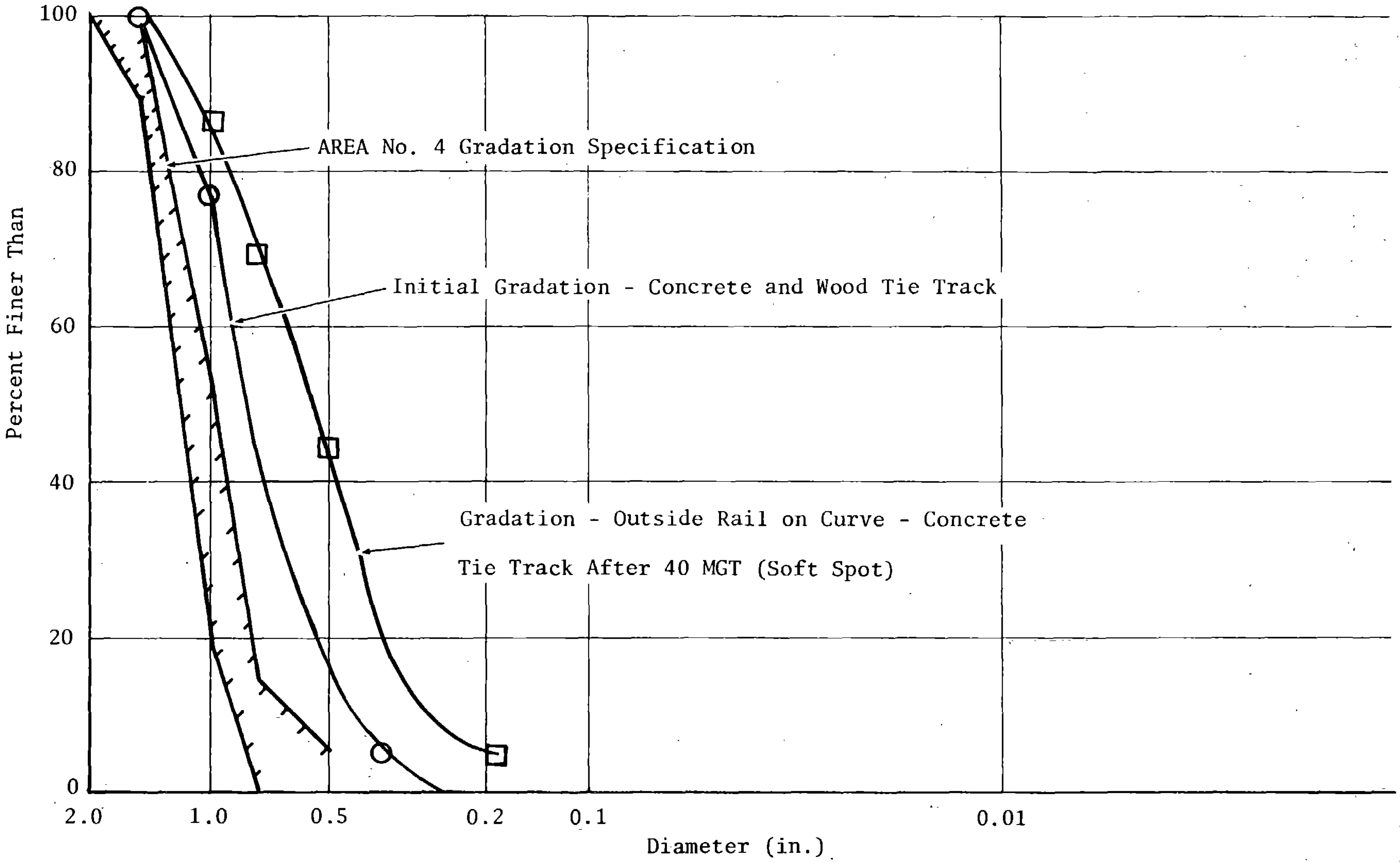


Figure 7-3c. Ballast Degradation Comparison - Concrete vs. Wood Tie Track

for these experiments (18 and 20) are shown in figure 7-4. The ballast depths vary from 11 to 23½ inches, which, when combined with the average subballast depth of 6 inches, produce a total depth of materials of 17 to 29½ inches. The types of ballast studied are granite, limestone (two sources), traprock, and blast furnace slag.

Measures that were used to compare component performance were changes in vertical profile, subgrade deformation, visual observations, ballast deformation, and ballast gradation.

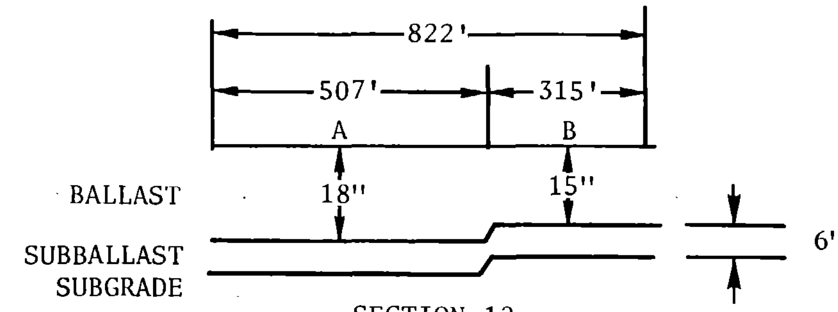
Sections 18 and 20 have jointed rail throughout and were newly constructed. Segments which included instrumented test sections were 18A, 18B (both granite), 20B (limestone), and 20E (traprock). Sections 15 and 22, existing track with jointed and continuously welded rail, respectively, were used in some analyses as a check on the significance of the value of the measurement of track performance.

7.2.1 Vertical Profile

Changes in track geometry were measured by survey to benchmark every 100 feet around the loop. Sections were not long compared to the distance between survey points, so the absolute and mean rail settlements as shown in figures 7-5a through 7-5d have little meaning taken out of context. Combining sections of like ballast depth and type, however, provide an accumulation of data points that begin to become significant.

Table 7-1a indicates the values for mean rail settlement and the standard deviation of the mean rail settlement (a measure of track roughness) for different ballast depths. Mean rail settlement appears to increase as ballast depth increases, but no trend in track roughness is evident with respect to variation in ballast depth. The relation of initial settlement to ballast depth has been observed elsewhere and is consistent with the notion that most

- 136# Jointed Rail
- OAK TIES
- 19-1/2 C-C TIE SPACING



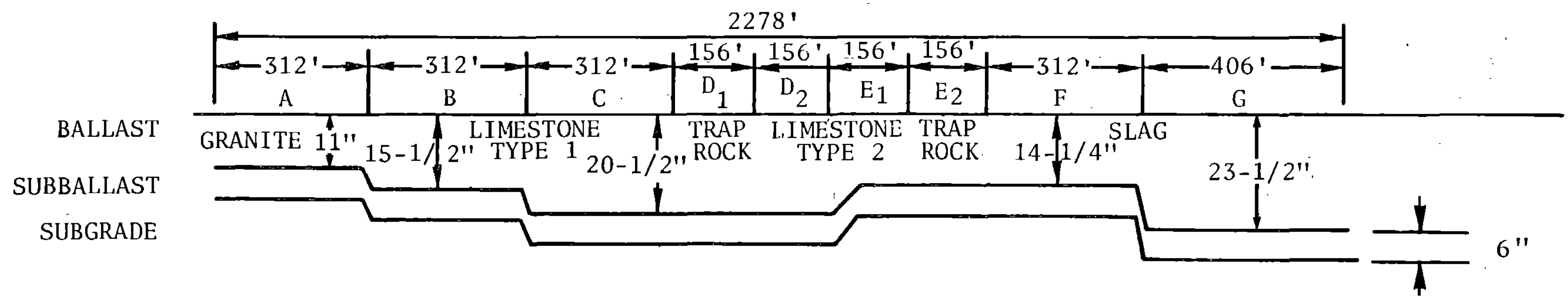


Figure 7-4. FAST Ballast, Depth of Sections

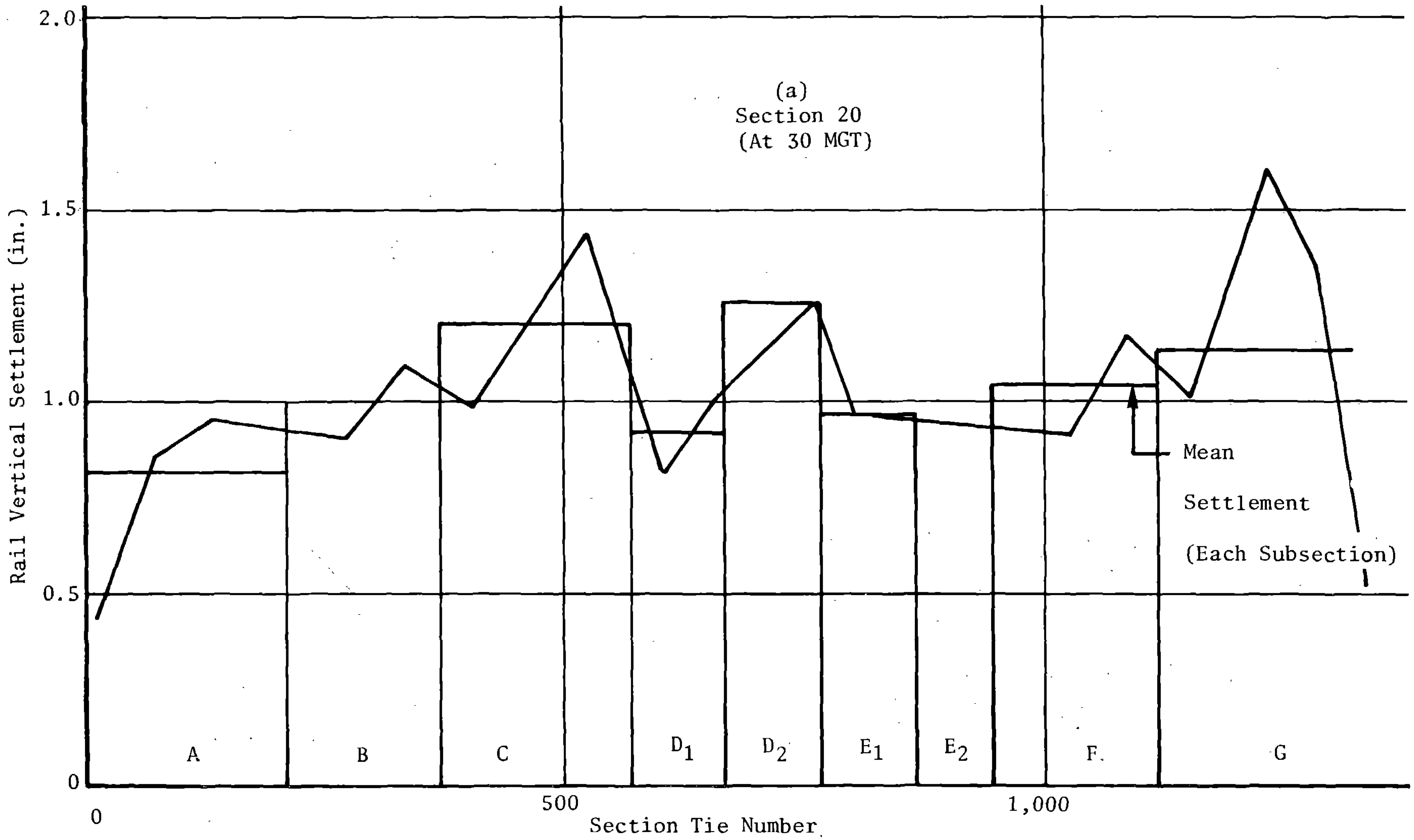


Figure 7-5. Vertical Rail Settlement (1 of 3)

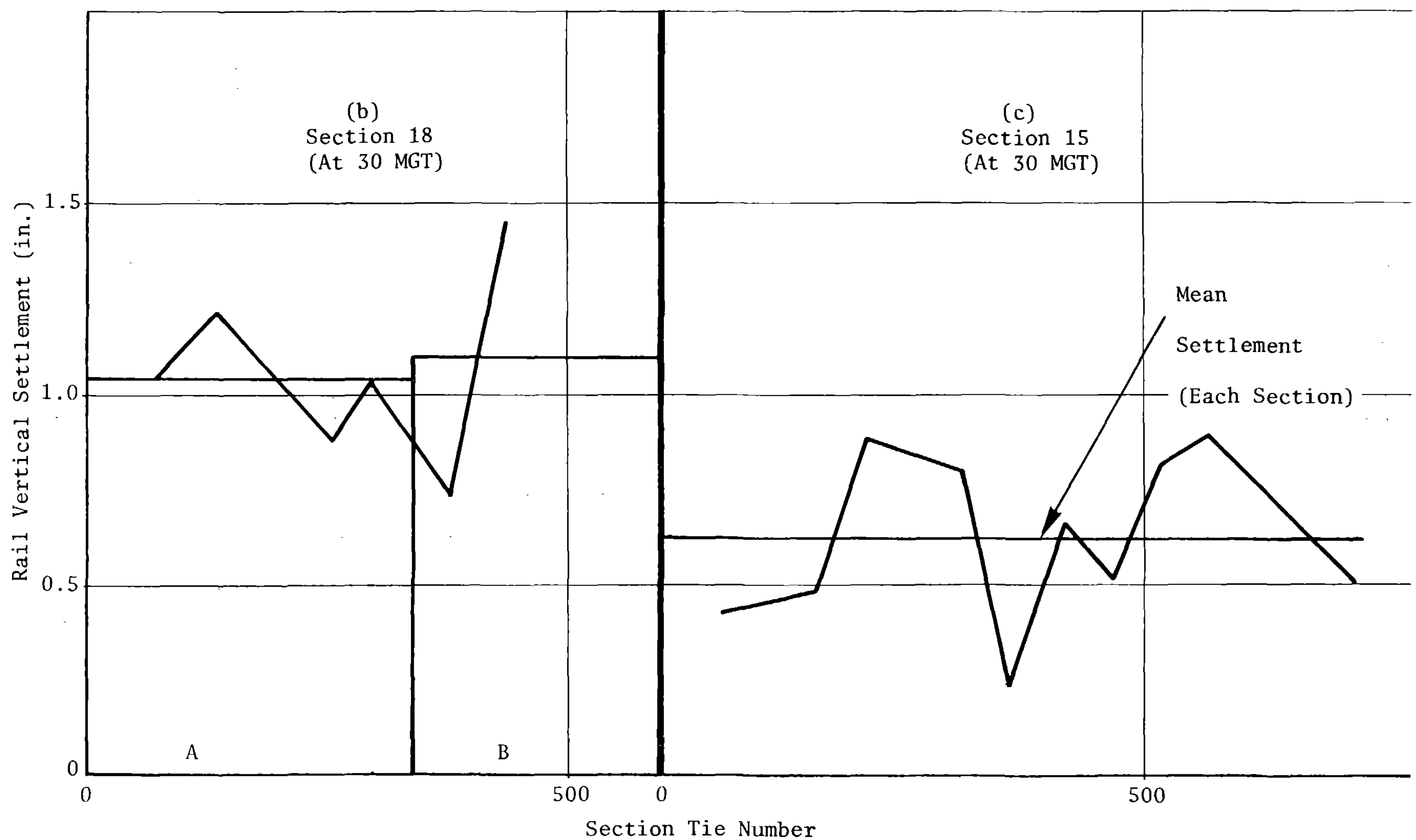


Figure 7-5. Vertical Rail Settlement (2 of 3)

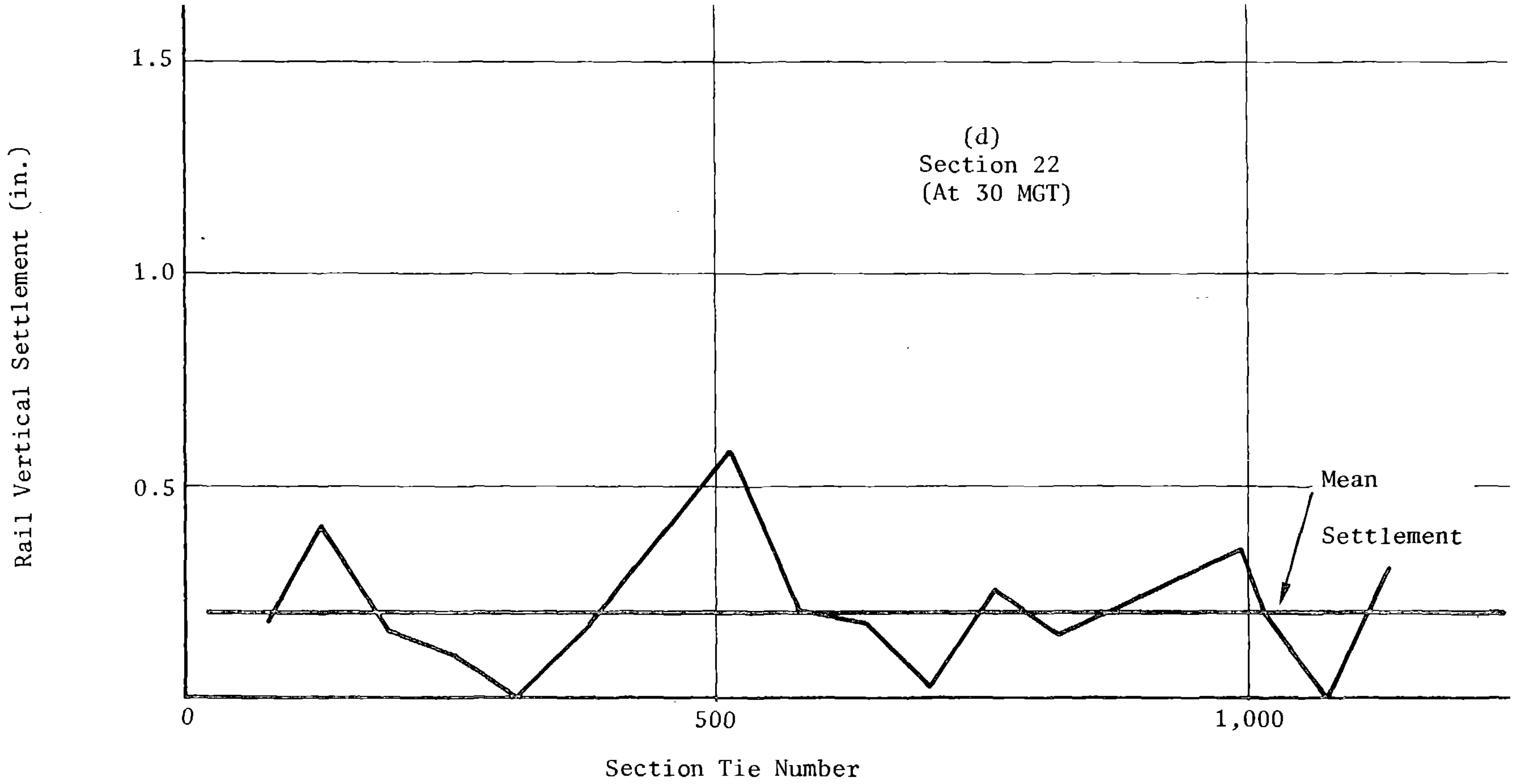


Figure 7-5. Vertical Rail Settlement (3 of 3)

initial settlement is due to ballast consolidation--the greater the ballast section depth, the greater the initial settlement. The differences in absolute settlement between different section depths, however, is not great.

The effect of ballast type on mean rail settlement and track roughness is even more ambiguous. Table 7-1b shows the values of mean rail settlement and its standard deviation for different depths of three types of ballast. No trend is apparent.

Having a significant impact on mean rail settlement and track roughness (as evidenced in table 7-1c) and, in fact, a significantly greater effect than variation in ballast depth is the type of track and the construction history of the embankment (at least for the FAST track embankment material). The observed mean rail settlement decreases on an existing roadbed as compared to a newly constructed roadbed, and again, a larger decrease in mean rail settlement can be observed in continuously welded rail track compared to jointed rail track. Perhaps more importantly, overall track roughness definitely decreased going from new construction to old construction (jointed rail) to existing CWR track. All the existing track was tamped prior to commencement of FAST traffic, as was the new construction, so that at least the top portion of the ballast all started from relatively the same compacted state.

7.2.2 Subgrade Deformation

Comparisons were made of subgrade deformations beneath varying ballast/ subballast section depths. Measurements were made in the same manner as described in section 7.1.2 using inductance coil extensometers. As indicated in figure 7-6, no real difference was found in subgrade deformation between ballast/ subballast depths of 18 to 27 inches.[3] One would not expect to see significant differences in subgrade deformation in this situation because of the type and quality of the subgrade and the rather deep ballast depths. However, as reported

Table 7-1a. Effect of Ballast Depth on Rail Profile

Inches		
Mean Ballast Depth	Mean Rail Settlement	Standard Deviation of Rail Settlement
11	0.75	0.23
15	1.02	0.22
21	1.07	0.21
24	1.13	0.44

(At 30 MGT)

Table 7-1b. Effect of Ballast Type on Rail Profile

Inches		
Ballast Type (depth)	Mean Rail Settlement	Standard Deviation of Rail Settlement
Granite (15)	1.09	0.42
Granite (21)	1.04	0.43
Limestone (15)	0.93	0.40
Limestone (21)	1.06	0.25
Slag (15)	1.04	0.15

(At 30 MGT)

Table 7-1c. Effect of Track Type on Rail Profile

	Inches	
Track Type	Mean Rail Settlement	Standard Deviation of Rail Settlement
New Construction - Jointed Rail	1.02	0.28
Pre-aged Construction* - Jointed Rail	0.63	0.21
Pre-aged Construction* - Continuously Welded Rail	0.20	0.15

(At 30 MGT)

*Existing Roadbed (0.5 MGT), Three Winter Cycles, Newly Tamped Ballast

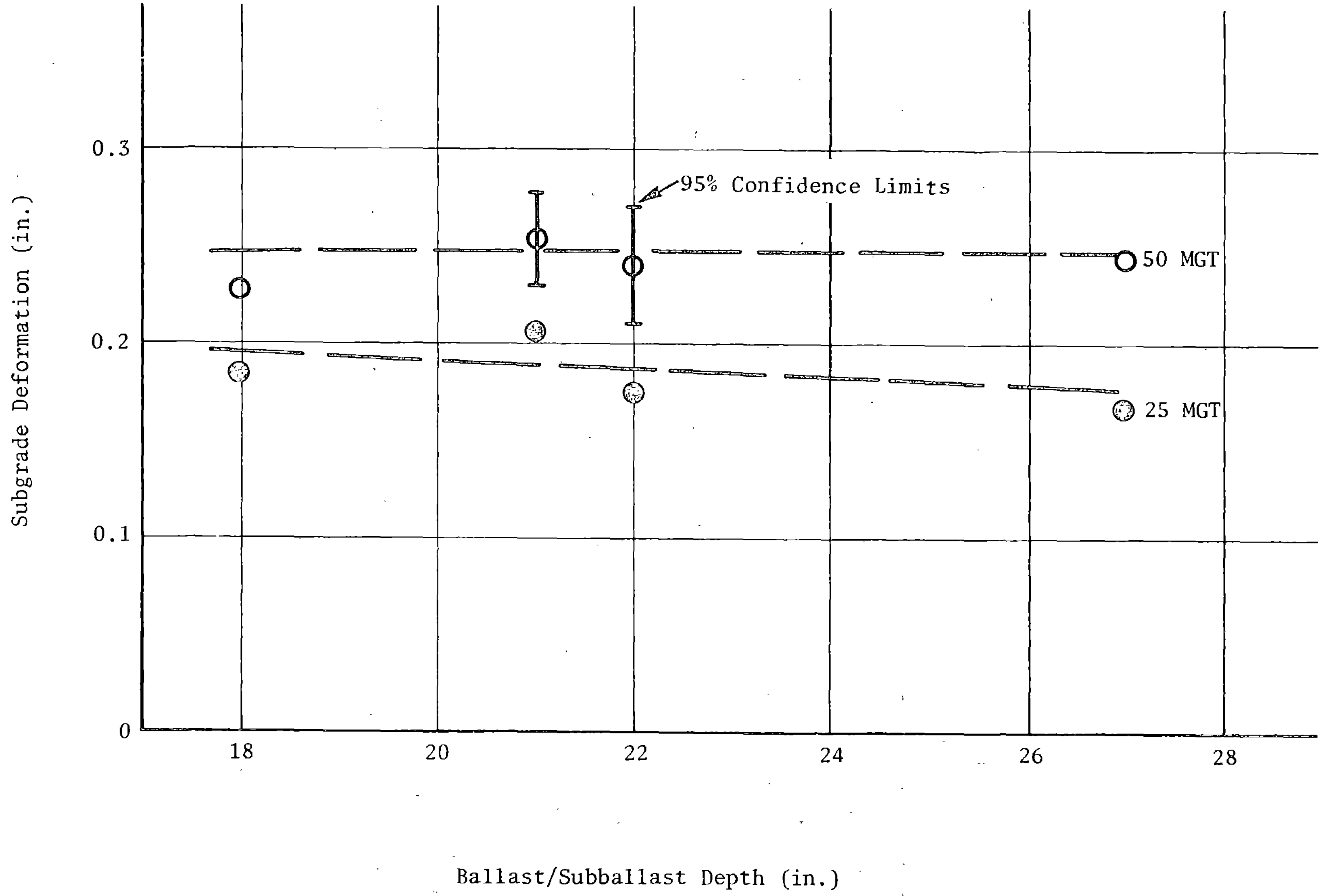


Figure 7-6. Variation in Subgrade Deformation with Ballast Depth

in the following section, at least one case was observed of a substantially thinner ballast section which showed signs of distress.

7.2.3 Visual Observations

One tie each was pulled in sections 20A and 18B in order to visually observe the condition of ballast after 40 MGT of accumulated traffic load. In each case, a slightly center bound tie was chosen to try to observe a "worst" case condition. Ballast samples for sieve analysis were taken and care was taken not to disturb any possible layer interfaces and especially the ballast/subballast surface.

Under the 7½-inch ballast depth section, figure 7-7a, a zone of fouled ballast was found which was quite moist. The subballast surface was taken to be the compact layer of finer particles that was found just below the predominately ballast sized particles. This layer was quite distinct and hand measurements showed approximately 1 inch of rutting in the subballast layer below the tie rail seat area.

Under a 15½-inch ballast depth, no rutting was observed, but intrusions of damp (meaning apparently less moist than the 7½-inch section) fines could be seen in the approximate pattern shown in figure 7-7b up to within 6 inches of the bottom of the center of the tie.

These observations were made in "soft" spots possibly atypical of the remainder of the wood tie track; however, they would seem to indicate that the appropriate ballast design depth for this track and this existing subgrade is closer to 15 inches than to 8 inches.

7.2.4 Ballast Deformations

Measurements of ballast deformation within the top 1 foot of ballast (see section 7.1.3 for discussion of identical measurement techniques) as shown in figure 7-8, show that although the limestone ballast has a larger initial

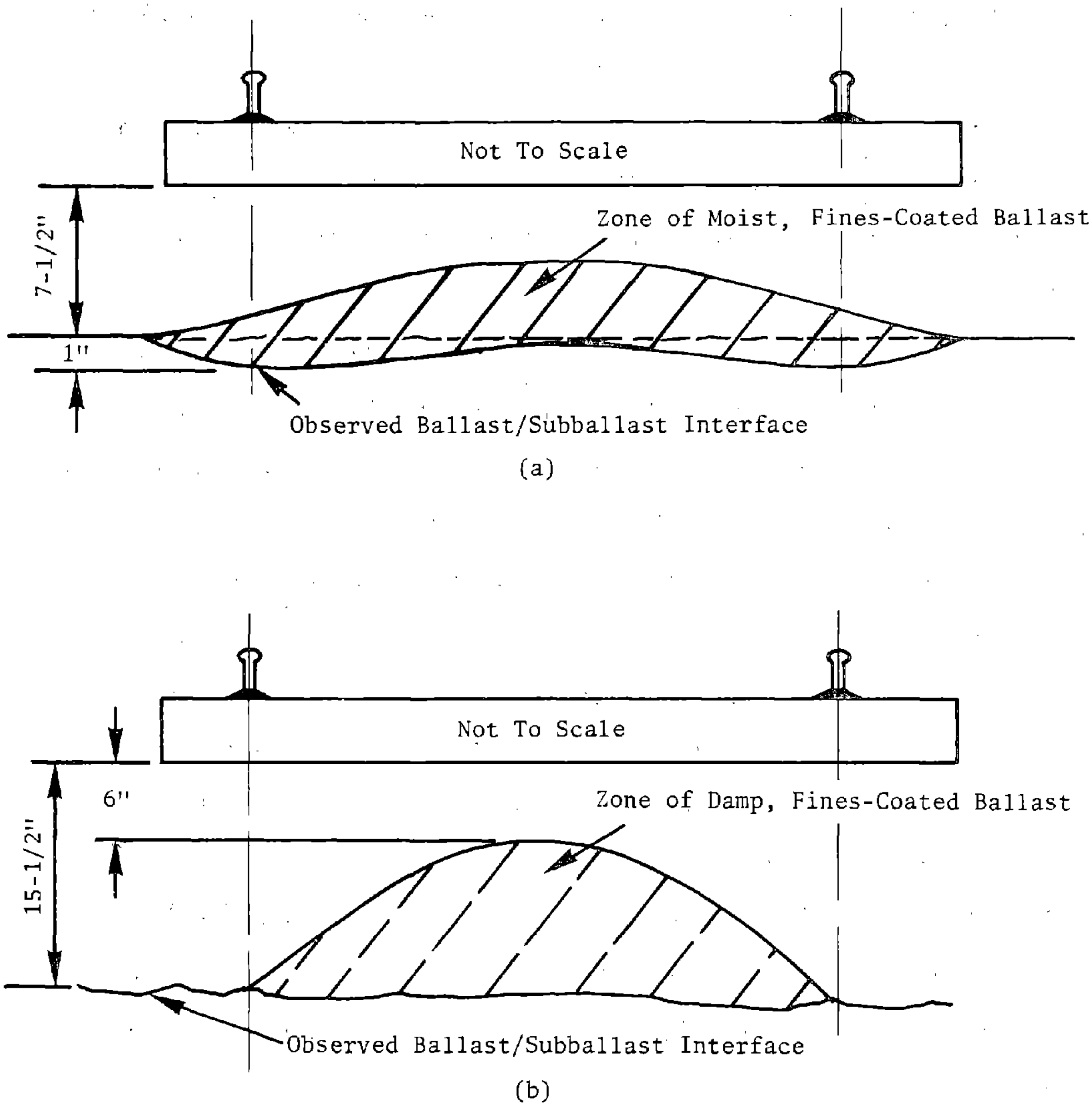


Figure 7-7. Visual Observation of Ballast and Subballast Beneath Two Ties

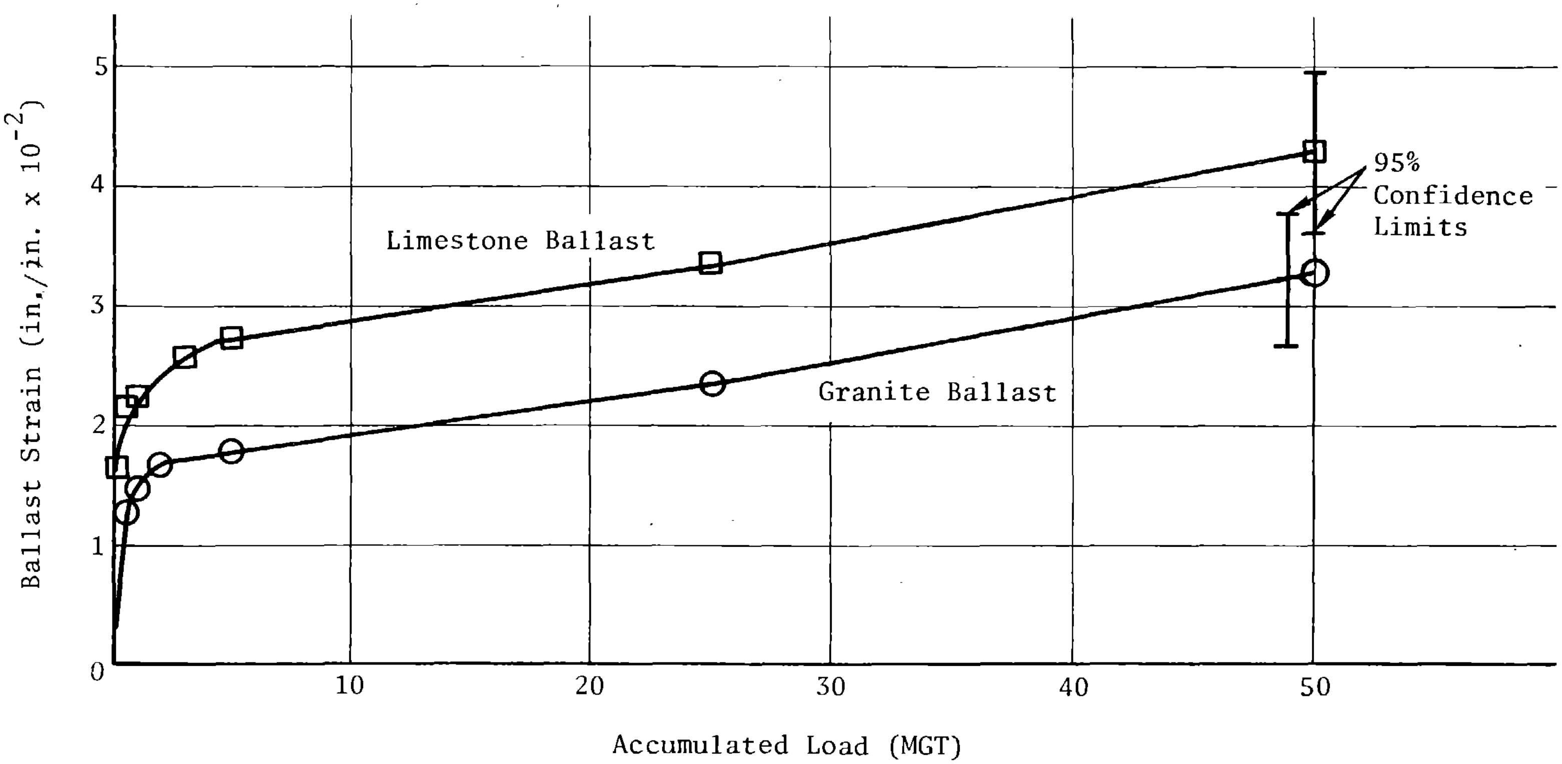


Figure 7-8. Comparison of Ballast Strains

deformation, the mean rate of deformation for both limestone and granite ballast, after approximately 5 MGT of traffic, is approximately identical. One explanation for this difference in initial deformation rate could be the finer particle composition of the granite ballast (see table 7-2) as compared to the limestone ballast. There is, therefore, probably no significant, discernible difference in the deformation properties of limestone and granite ballast within the first 50 MGT of traffic at FAST.

7.2.5 Ballast Gradations

Samples were taken from under ties in both the limestone and granite ballast sections and sieve analyses were run. As indicated in table 7-2, no differences in gradation could be observed. It must be recalled, however, that this track (and these ballasts) have only seen one winter with very limited freeze-thaw cycles and little precipitation. Some ballasts are very sensitive to the environment and the lack of visible degradation at FAST is not necessarily a good indicator of potential performance in another, more severe environment.

7.3 Ballast Shoulder Width Comparison

Ballast shoulder widths were varied in FAST in sections 3 and 15. The specific widths proposed for study were 6, 12, and 18 inches. Section 3 is the rail metallurgy test section and consists of continuously welded rail on wood ties on a 5^{o} curve. Shoulder widths used in section 3 were 6, 12, and 18 inches. Section 15 was the ballast shoulder width test section on tangent wood tie track with jointed rail and consisted of two sections, one with a 6-inch and the other with an 18-inch ballast shoulder width. Analysis for the purpose of this study was limited to section 15 and the measures of interest were rail alinement and horizontal track stiffness.

Table 7-2. Comparison of Ballast Degradation

(From Sieve Analysis of FAST Data)

PARTICLE DIAMETER (IN.)	GRANITE		LIMESTONE	
	INITIAL % FINER THAN	CHANGE IN % FINER THAN AT 40 - 50 MGT	INITIAL % FINER THAN	CHANGE IN % FINER THAN AT 50 MGT
1	77.2	-0.2	67.7	+1.0
3/4	45.8	+1.0	24.2	+1.0
1/2	18.8	-0.4	7.5	-3.6
NO. 4 SIEVE (0.187)	1.2	-0.7	2.3	-1.7

7.3.1 Rail Alinement

Lateral alinement of rails was measured using optical techniques. Due to the short test sections for ballast shoulder width at FAST and therefore the limited number of data points, no statistically significant trends could be observed.

7.3.2 Horizontal Track Stiffness

Horizontal track stiffnesses were measured as described in chapter 4 for both shoulder widths in section 15. Horizontal track stiffness was greater in the 18-inch ballast shoulder width section than in the 6-inch ballast section, but precise values of lateral load response are not complete and are statistically unreliable due to inconsistencies in the test procedure and unique characteristics of the test sites in section 15. Specifically, location of joints, exact plane of transverse loading, and even exact values of ballast shoulder width are uncertain. The application of the numerical results of these lateral pull tests to the evaluation of the performance of section 15, ballast shoulder width, is therefore not practicable, in this instance.

CHAPTER 8. SUMMARY OF FINDINGS

This chapter summarizes the findings, trends, and conclusions which were a result of the analyses performed for this report as well as results and general observations from other FAST track experiments through the first 50 MGT of traffic.

8.1 General FAST Track Results

The steel tie fasteners exhibited early fatigue problems which forced early replacement of the ties at about 27 MGT with wood ties. An inspection of the steel ties in revenue traffic also showed the same failure mechanism. Both groups of steel ties had been subjected to approximately the same MGT; however, the time spans were different. This provided an early indication that some track components on FAST might experience a life cycle that could be compared to revenue conditions.

The original FAST switch points required considerable grinding and sometimes required replacement. This was especially true for those in diverging moves. Two manganese switch points were field installed at 30 MGT. This was the first time a field application had been attempted. The existing top surface of the point was ground off and the new point bolted to the movable point. While grinding is still necessary, but not with the magnitude earlier required, the manganese tips are exhibiting far greater life as compared to the standard steel points.

Many other aspects of the FAST track at this early MGT have not provided direct conclusions. However, statements can be made which relate to the fact that many tracks in revenue service exhibit similar wear patterns. Several examples follow to illustrate that FAST track performance is not all too different than revenue service situations.

FAST rail wear varies considerably around the loop. Some of this was due to the nonuniformity of rail lubrication. Many other items also affect rail wear, but for FAST the fact that all wheels at the beginning had the same, new profile was a significant contributing factor in rail performance. When FAST wear data and rail profiles are compared to rail data from unit train operations, similarities can be found. Improved rail lubrication and testing of rail of various metallurgies on all curves of FAST will add data which will provide the basis for significant conclusions.

Rail corrugations were observed on the low rail on all of the curves. Corrugations exist in revenue service in conditions not totally unlike those of FAST. Data are now being collected to determine if any correlations exist between other test variables and the amplitude, wavelength, and location of rail corrugations. The rail will be ground as soon as all the necessary data are collected. Future tests will be made to verify any trends resulting from the data analysis.

FAST special track work, consisting of turnouts, frogs, insulated joints, and compromise joints, have required more maintenance than the track sections. Maintenance has been mainly in the form of tamping to maintain track geometry and uniform support conditions. This is comparable to revenue service. Special track work is usually subjected either to greater impact loads or the component parts are not as strong as or as load resistive as normal track. Therefore, this extra maintenance could be expected.

8.2 Concrete and Wood Tie Track Systems Performance Summary

The trend from the data on newly constructed concrete tie versus wood tie track within FAST indicates a larger amount of settlement of concrete tie track versus wood tie track. It appears that concrete tie track settled faster in the first 20 MGT and that from 20 to 40 MGT concrete and wood tie track settled at approximately the same rate. This result is supported by results from the ORE Track Test Loop at Velim through 33 MGT.

In contrast to the results on a new embankment, the evidence at Velim for concrete tie track built on an existing roadbed, showed considerably less settlement. This suggests that the embankment and ballast conditions for new and rebuilt track may have a more significant effect on overall settlement than tie type.

This does not appear to be the case though, when evaluating uniformity in track settlement. There is a difference between concrete and wood tie track in differential settlement which may be attributable to the tie type. Concrete tie track appears to be settling more uniformly, and as a result, the surface and profile geometry should be better than that of wood tie track. As yet, however, the track geometry data at FAST has not been conclusive enough to bear this out. More study of the future track geometry data at FAST is needed to verify this trend.

The use of track geometry data at FAST to evaluate the performance of concrete versus wood ties was limited to an evaluation of gage. The data to date from FAST indicates that concrete tie track has slightly better gage capabilities as compared to wood tie track.

Other data on track alinement errors from European and domestic railroads show that concrete tie track has a lower rate of geometry deterioration than comparable wood tie track.

The results of the settlement and track geometry data are borne out by the results of the vertical and horizontal track moduli tests, where the concrete tie track was a good deal stiffer than the comparable wood tie section. The stiffer section should result in better track geometry retention, and it should, for the greater lateral resistance shown, provide greater stability for continuously welded rail.

The performance of concrete tie track is affected by a major consideration, track maintenance and construction. From the evidence of data to date and from observations of the track maintenance and construction, equipment and operations should be modified from that used for wood tie track. The concrete tie is heavier, has special fasteners, and will not withstand large center-bound loading conditions. Therefore, the normal track construction and maintenance equipment may require some modification or may need to be more substantial in order to perform adequately on concrete tie track.

Also to be considered are the quality control aspects of each component, which must be carefully monitored, to be assured of an adequate functioning system. In a sense, this applies to other aspects of the FAST track as well. The amount of maintenance on FAST track sections has varied from none to more than was planned. For example, many field welds have failed or have been removed following ultrasonic inspections. To date, a total of 45 welds have been replaced, consisting of 13 shop welds and 32 field welds. Seven of the 45 occurred in the concrete tie track section. Shop welds of similar rail metallurgy have experienced only one failure. Due to a lack of available resources, there were instances where continuous welded rail track had bolted joints for as much as 15 MGT before new field welds could be completed. As a result of the joints in CWR, some FAST sections experienced conditions which created abnormal loads to track components and added maintenance requirements. This is indicative of a need to have good quality control on field welding.

8.3 Tie and Fastener Component Performance Summary

It is evident that even at FAST, with its higher average occurrence of high vertical wheel/rail loads, the bending moments for the concrete ties at the rail seat and at the tie center are low in comparison with the ultimate strength of the tie, but they compare well with data available from other test tracks.

This would suggest that a possible failure mechanism for concrete ties might be fatigue rather than peak loads. Since FAST has only 50 MGT to date and the incidences of wheel flats at FAST is small, a continued assessment of the performance in this area at FAST and other test sites is needed to verify the above statement.

In any case, the bending moments on the concrete ties at FAST are well within those required for the tie by the AREA Specifications.

As can be expected from this evidence and from the low MGT levels to date, there are no ties that have structurally cracked due to the service conditions at FAST.

One measure of performance that indicated a condition that would affect the maintainability of the concrete tie track was tie movement. This tie movement was symptomatic of the severe conditions created at FAST by the use of rail plugs at the point of field weld failures and by the existence of rail corrugations on the low rail in curves; both of which, it is believed, caused several soft spots to occur. These soft spots were the points where the ties were moving excessively, much as wood ties do near bolted joints. It is evident from FAST that the existence of jointed track and large rail corrugations in concrete tie track should not be tolerated.

The performance of the concrete tie fastener system, clip, pad, and insulator has been satisfactory. There has been some insulator movement and cracking, and some fasteners have been loosened, but as yet this has not been a critical performance or maintenance problem.

The spring type fastener and tie plate on wood ties appear to reduce the tie plate cutting when compared to cut spikes and standard tie plates. However, due to the shortness of the test section and the test period, no other substantive

FAST data on the fastener system ability to restrain the rail longitudinally and laterally are available.

8.4 Rail Wear Comparison Summary

The data analysis indicated that there were some trends evident in rail wear at the gage point on the FAST track.

The chrome molybdenum and head hardened rail appear to be wearing the least, though there is no significant difference between the wear on those two types of rail.

There is a trend in the data that shows that rail wear is greater on the 1:14 cant tie plate. This trend is not conclusively proved from data collected up to 40 MGT and may only be indicative of the wear pattern on the rail as a result of the severe cant and change in the wheel/rail contact surface from that of the 1:40 and 1:30 cants.

There was slightly less wear on standard rail in section 17 (concrete ties) than in section 3 (wood ties). The difference could easily be attributable to the nonuniformity in rail lubrication. Further analysis will be required as more wear cycles on the FAST track are completed.

8.5 Ballast and Subgrade Performance Summary

The results of analyses of the performance to date of the ballast and subgrade in concrete tie track as compared to wood tie track, show that concrete tie track--even though it loads the subgrade to the same level as wood tie track--deforms the ballast and subgrade to a greater extent than wood tie track. This may be attributable to the loading mechanism of concrete tie track which appears to "punch" into the ballast.

Whether or not this will continue to occur, will happen on existing rebuilt track, or will occur with different ballast and subgrade materials is not known. However, the trends here support the results as reported in chapter 4, and if the trends continue as suggested, then the rate of settlement in the track will decrease and the rate of settlement between concrete and wood tie track will become more equal.

As a result of the ballast gradation analysis it is evident that the concrete ties will initiate ballast degradation in a granite ballast where high impact loads are transmitted to the ballast because of a "soft" spot. This reinforces the result, as stated previously, that the occurrence of rail corrugations or the use of rail plugs in concrete tie track will lead to progressive deterioration of the track.

As of 50 MGT, there is little evidence of a trend in the performance of the different types of ballast and the depth of ballast at FAST. It is anticipated that later phases of FAST will provide more input into this area, but much more analysis remains to be done at greater MGT levels, especially in comparing the performance of the various ballast types. Much of the same can be said for the evaluation of the variations in ballast shoulder width.

APPENDIX A

CONCRETE TIE TEST: SECTION 17

Test Section Configuration

The concrete tie test section is 6,143.31 feet in length (see figure A-1) and involves 17 unique combinations of 6 types of ties, 9 types of pads, and 3 types of fasteners.

FAST section 17, concrete ties and tie pads, is subdivided into 15 test segments (see figure A-2). Rail consists of field welded strings of standard 136-pound rail. The ballast consists of approximately 15 inches of crushed granite under the ties and 12 inches on the shoulders. Ties are spaced 24 inches center-to-center. Table A-1 indicates the various tie, pad, and fastener combinations and locations within section 17.

Rail

Field welded strings of standard 136-pound CWR were used previously on the Kansas Test Track (KTT). At the KTT, on tangent track, the rail was exposed to about 20 to 25 MGT of main line traffic over a period of 6 months.

Ties

Six types of ties, all of which are laid at 24-inch centers, are used in the concrete tie test section. Except for the T-3 tie, all meet the requirements of "Preliminary Specifications for Concrete Ties (and Fastenings)," American Railway Engineering Association - Bulletin 655, and as modified in Bulleting 660. Table A-2 identifies the principal physical characteristics of the concrete ties.

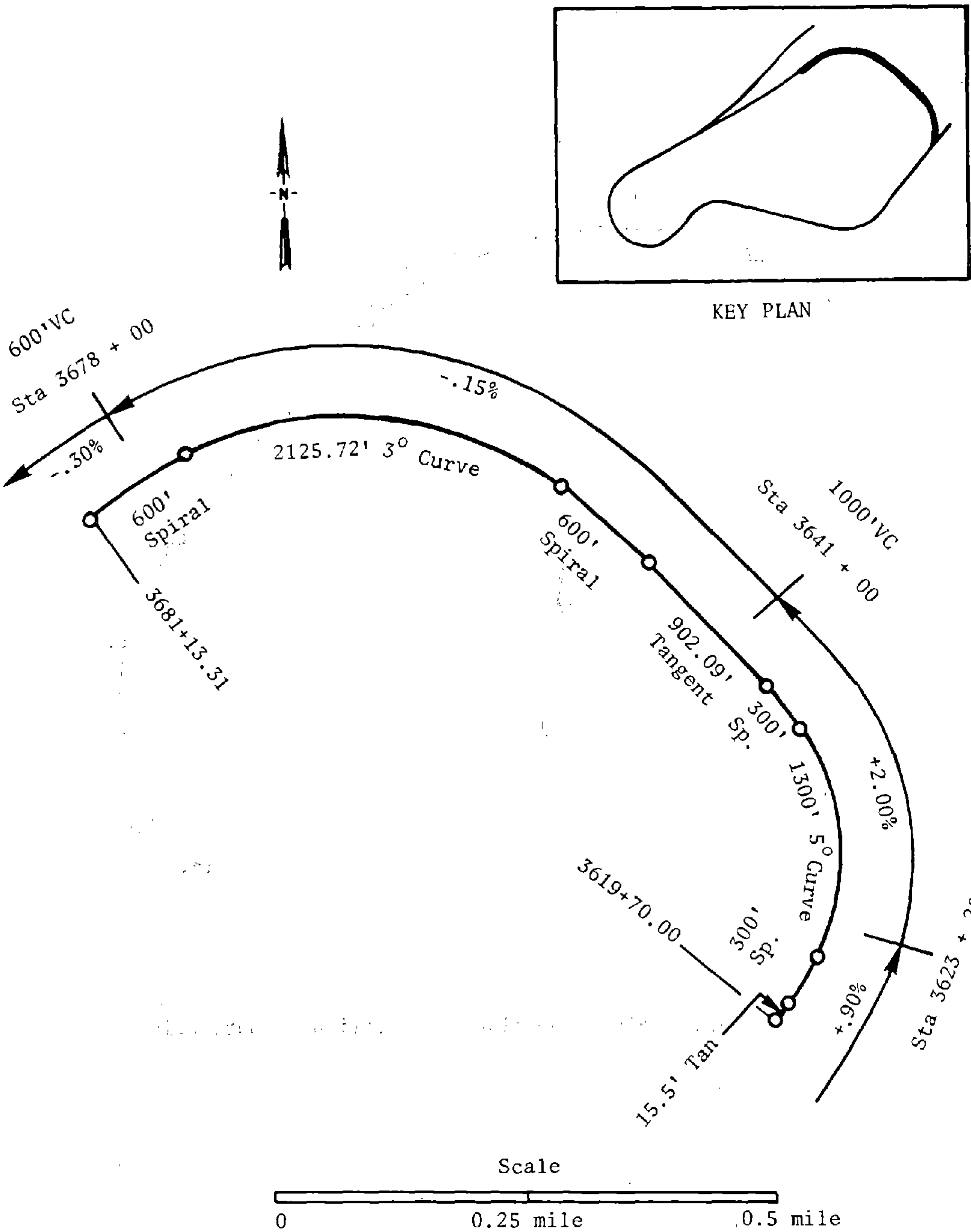


Figure A-1. FAST - Section 17, Horizontal and Vertical Alinement

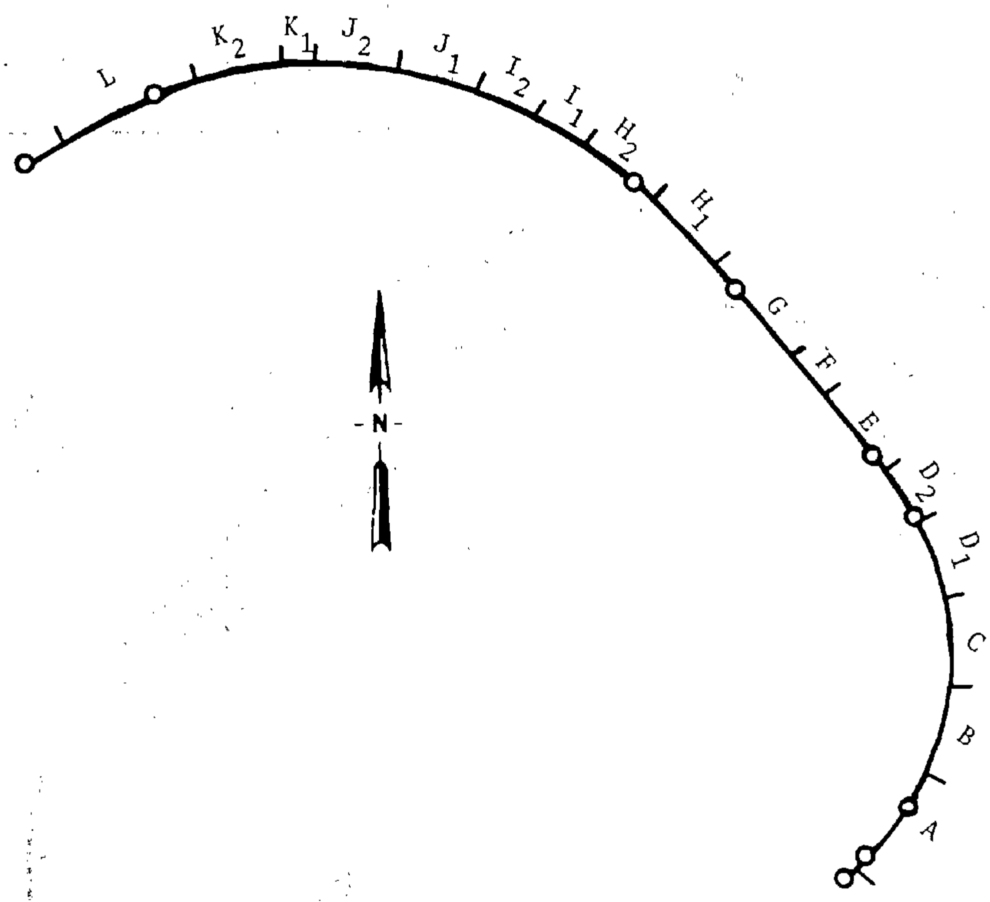


Figure A-2. FAST - Section 17, Location of Subsections

Table A-1. Section 17 - Track Components

SUB-SECTION	TIE	PAD	FASTENER	AS-BUILT LENGTH (ft.)	AS-BUILT NUMBER OF TIES	NUMBERING OF 1ST TIE
TRANSITION SECTION	WOOD, 8'-6"			29	21	17-0001
	WOOD, 10'-0"			93	50	-0020
A	T-1	P-1a, b	F-1	529	260	-0070
B	T-2	P-1b, a	F-1	325	159	-0330
C	T-1	P-2	F-1	326	162	-0489
D_1	T-2	P-2	F-1	317	158	-0651
D_2	T-2	P-2	F-2	209	101	-0809
E	T-1	P-3	F-1	396	198	-0910
F	T-3	P-4	F-3	202	99	-1108
G	T-4	P-2	F-1	510	252	-1207
H_1	T-5	P-5	F-4	355	175	-1459
H_2	T-5	P-5	F-1	333	171	-1634
I_1	T-4	P-1a, b	F-1	252	123	-1805
I_2	T-4	P-6	F-1	257	127	-1928
J_1	T-1	P-6	F-1	409	203	-2055
J_2	T-1	P-7	F-1	310	154	-2258
K_1	T-2	P-7	F-1	136	67	-2412
K_2	T-2	P-2	F-1	364	177	-2479
L	T-6	P-8	F-1*	600	300	-2656
TRANSITION SECTION	WOOD, 8'-6"			200	124	-2956 17-3080

*Supplied with T-6 - Slightly Different than F-1

Table A-2. Tie Demographic Data

Tie	Length (feet)	Weight (pounds)	Prestress Steel	Fastener
T-1	8½6	721	28 - 5 mm dia. wire	F-1
T-2	8½	745	8-3/8" dia. strand	F-1 and F-2
T-3	9	680	6-3/8" dia. strand	F-3
T-4	8½	760	7-3/8" dia. strand	F-1
T-5	8½	860	8-3/8" dia. strand	F-1 and F-4
T-6	9	850	22 - 0.196" dia. wire	F-1*

Tie Pads

Nine types of pads are used with the concrete ties. The pads primary function is to isolate the tie from direct rail contact thereby cushioning the load to the tie.

Two of the pads, P-1a and P-1b, are manufactured from a high density polyethylene material with an operating temperature of -150° to +200° F, while the other material has an operating temperature of -150° F to +150° F. The physical data for each material is very similar except for their resistance to cold flow under pressure. The operating temperature describes the range of resistance of each material to cold flow.

P-2 pads are made of corded rubber from heavy-duty truck tire fabric. Those used with the T-4 ties were supplied as left-hand and right-hand shapes. This is necessitated by the unsymmetrical arrangement of fastener shoulders and the fact that the pads must always be placed with the convex side facing up. Only one shape is required for the other pads, since the pads may be turned on either side. The P-2 pads for use with the insulated shoulder, fastener F-2 on the T-2 tie, must have a thinner section (6 inches rather than 6¼ inches) between the inserts.

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*An earlier version of fastener 7-1 for use on tie T-6 only.

The P-3 tie pads, used in the concrete tie test section are 3/16 inch thick and are made from a rubber base reinforced with cotton fiber. The 3/16-inch Polyurethane tie pads (P-5) are used with the T-5 ties for both the F-4 and the F-1 fastener shoulders.

The P-6 tie pad is manufactured from a type 6 nylon to which carbon black has been added for ultraviolet light stabilization. This 3/16-inch thick pad is for use with F-1 type fasteners on a 6-inch rail base. The P-7 pad is a 3/16-inch thick synthetic rubber, grooved tie pad. The 3/16-inch thick P-8 polyethylene tie pads were supplied with the T-6 concrete ties.

Rail Fasteners and Insulators

Three types of fasteners, whose primary purpose is to hold the rail onto the tie, are used on the concrete tie test section. They are noted as F-1, F-3, and F-4. On the fastener system F-2, the only difference from the F-1 fastener is the insulated shoulder.

The F-1 rail clip is an elastic fastener which provides both longitudinal and lateral restraint to the rail and meets the requirements of American Railway Engineering Association - Bulletin 655 and as modified in Bulletin 660. In most sections, the fastener is used with a noninsulated shoulder and insulator, exception in section D_2 where the insulation is provided by an epoxy coating on the shoulder insert which changes the system to the F-2 designation.

The F-3 rail fastener is a bolted fastener, which was modified for use on the Kansas Test Track. The F-4 clip and shoulder is an elastic fastener designed to meet the specifications of Bulletin 655. Insulation is accomplished with an epoxied shoulder.

Measurements

Measurements for this test include track geometry, survey to benchmark, rail profile, rail creep, tie insulation, ballast gradation, tie movement, tie pad movement, tie fastener and insulator movement, concrete tie flaw inspection, pad performance, vertical track stiffness, horizontal track stiffness, ballast strains, ballast/subballast pressures, subgrade strains and vertical wheel/rail loads.

RAIL METALLURGY TEST: SECTION 3

Test Section Configuration

The rail metallurgy test section analyzed consists of 3,740 feet of track, 3,672.7 feet of which is a 5° curve and 67.3 feet of a 300-foot spiral.

Rail

The rail in this section consists of 20 field welded 374-foot strings of 132- to 136-pound plant welded CWR. Each rail string consists of 78-foot sections of head-hardened, high silicon, fully heat-treated, and chrome molybdenum rail and a 62-foot section of standard rail.

Tie Plates and Ties

Three different types of tie plates are being tested for their effect on rail wear and tie plate cutting. They are the standard 7-3/4 x 14-inch, 1:40; 7-3/4 x 14-inch, 1:14; and 8-1/2 x 16-inch, 1:30. Oak ties, 7 by 9 inches by 8½ feet, are used throughout this section.

Measurements

The measurements used were rail profile, survey to benchmark, and track geometry car data.

RAIL TIE FASTENERS TEST: SECTION 7

Test Section Configuration

FAST section 7, rail tie fasteners, is a 1,000-foot test section on a 5^{o} curve. The test section is divided into five segments, each 200 feet long. This test section is built on existing track with the following fasteners:

Segment	Rail/Tie Fasteners	Rail Anchors	Tie Plates (inches, cant)
A	4 cut spikes per plate	Used, box anchor every other tie	7-3/4 x 14 1:40
B	2 cut spikes at rail, 2 lock spikes on plate	Box anchor every other tie	7-3/4 x 14 1:40
C	2 cut spikes on plate, 2 compression clips at rail	None	7-3/4 x 14 1:40
D	Elastic, 2 lock spikes on plate	None	7-3/4 x 15 1:40
E	2 cut spikes at rail, 2 screw spikes on plate (100 feet with double coil washer)	As in segments A and B	7-3/4 x 14 1:40

Rail

Rail in this test section is KTT 136-pound CWR and has been exposed to 20 to 25 MGT of traffic over a 6-month period. The rail was transposed after 33 MGT at FAST.

Ties

Ties are 7- by 9-inch oak, 8½ feet long on 19½-inch centers.

Measurements

The measurements used were tie plate cutting and rail profile.

BALLAST TYPES AND DEPTHS TEST: SECTIONS 18 AND 20

Test Section Configuration

FAST section 20, ballast types and depths, is a 2,278-foot tangent track section divided into nine segments. Constants in this test section are 136-pound jointed rail, standard tie plates with four spikes per plate, 7- by 9-inch oak ties, 8½ feet long on 19½-inch centers, and rail anchors 16 per rail length.

Ballast Types and Depths

Segment	Length (feet)	Depth (inches)	Material
A	312	11	Granite
B	312	15	Limestone Type 1
C	312	21	Limestone Type 1
D_1	156	21	Traprock
D_2	156	21	Limestone Type 2
E_1	156	14	Limestone Type 2
E_2	156	14	Traprock
F	312	14	Blast Furnance Slag
G	406	24	Blast Furnance Slag

FAST section 18, ballast depth, is an 822-foot tangent track test section divided into two test segments. Constants on this section are 136-pound jointed rail on 7- by 9-inch oak ties, 8½ feet long on 19½-inch centers; standard tie plates with four spikes per plate; and used rail anchors, 16 ties per rail length.

Ballast Depth

Segment	Length (feet)	Depth (inches)	Material
A	507	18	Granite
B	315	15	Granite

Subgrade

The subgrade is composed almost entirely of a cementing sand.

Measurements

Measurements for this test include track geometry, ballast gradation, survey to benchmark, ballast strains, subgrade strains, ballast/subgrade pressures, vertical track stiffness, and visual inspections.

BALLAST SHOULDER WIDTH TEST: SECTION 15

Test Section Configuration

FAST section 15, ballast shoulder width, is a 1,300-foot tangent track test section divided into two segments. This test section is built on existing track with the ballast shoulder altered in different areas. The ballast shoulder was reduced from 12 inches to 6 inches for 550 feet and increased to 18 inches for another 550 feet.

Ballast Shoulder Width

Segment	Length (feet)	Width (inches)
A	550	6
B	550	18

Ballast

The ballast in this test section, blast furnance slag, was initially laid in 1975, and the traffic prior to the initiation of FAST is estimated to be 0.5 MGT. The ballast shoulder width was initially 12 inches. The ballast depth is 15 inches.

Rail

Rail is 136-pound jointed rail.

Ties

The test section consists of 7- by 9-inch oak ties, 8½ feet long on 19½-inch centers.

Tie Plates and Fasteners

Tie plates on section 15 are standard, with 4 cut spikes for each plate.

Measurements

Measurements for this test were horizontal track stiffness and track geometry car data.

MEASUREMENTS

Track test data are collected at the TTC and then transferred to tape in digital format to provide off-line processing and a permanent record of the tests. These data were processed into a more useable format at TTC for use in this report.

Track static measurements were made in accordance with a startup plan, and except as noted, subsequent dynamic measurements were made at 25 MGT intervals and static measurements at 4-week intervals. The measurement plans are outlined in the FAST Test Specification.[1]

The measurement methodology is also described in detail in the specification.

Several measurements are common to each of the above test sections. They are:

a. Track inspection

b. Track geometry

c. Rail flaw detection

d. Longitudinal rail stress

e. Survey to benchmarks

The following additional measurements are taken in one or more of the test sections:

a. Static Measurements:

(1) Rail profile

(2) Tie plate cutting

(3) Rail creep

(4) Tie insulation

(5) Ballast gradation

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[1]"FAST Test Specification #7134," Transportation Test Center, Pueblo, Colorado, September 1976 (with modifications).

(6) Tie movement

(7) Tie pad movement

(8) Tie fastener and insulator movement

(9) Concrete tie flaw inspection

(10) Pad performance

(11) Ballast strains

(12) Subgrade strains

(13) Visual track inspections

b. Dynamic Measurements:

(1) Vertical track stiffness

(2) Horizontal track stiffness

(3) Ballast strains

(4) Ballast/subballast pressures

(5) Subgrade strains

(6) Rail vertical loads

(7) Concrete tie strains

APPENDIX B. NOTES

General

"FAST Test Specification #7134," Transportation Test Center, Pueblo, Colorado, September 1976 (with modifications).

"FAST, The First Experiment," Association of American Railroads, Chicago, Illinois, August 1976.

"The FAST Track," U.S. Department of Transportation, Federal Railroad Administration, Washington, D.C., September 1976.

Chapter 4

[1]Donald P. McConnell, "Observations on the USSR Shcherbinka Test Track Facility," Proceedings of the International Railroad Technology Transfer Seminar, New Orleans, Louisiana, January 1977 (available from Federal Railroad Administration).

[2]Janin, et al, "Optimum Adaptation of the Conventional Track to Future Traffic; Study of the Change in Track Geometry as a Function of Traffic," ORE Question D 117, Report D 117/RP7, ORE, Utrecht, October 1975.

[3]G. H. Way, Jr., "Progress Report - Concrete Tie Track at Noble, Illinois," Report No. 73-102, prepared by C&O and B&O Railroad Company, March 1973.

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[6]Ta-Lun Yang, "Track Geometry Measurements and Track Standards," ICT, Los Angeles, California, July 1976.

[7]Janin, op. cit.

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[11]J. Lundgren and W. W. Hay, "A Simulation Model of Ballast Support and the Modulus of Track Elasticity," Civil Engineering Studies, University of Illinois, Urbana, Transportation Series No. 4, September 1970.

[12]McConnell, op. cit.

[13]E. G. Cunney, et al, "The Effects of Accelerated Ballast Consolidation," Federal Railroad Administration, Report No. DOT/ORD-76/274, March 1977.

Chapter 5

[1]American Railway Engineering Association - Bulletin 644, Volume 75, September-October 1973, and American Railway Engineering Association Bulletin 655, Volume 77, November-December 1975.

[2]American Railway Engineering Association - Bulletin 650, Volume 76, November-December 1974, and American Railway Engineering Association - Bulletin 660, Volume 78, November-December 1976.

[3]J. W. Weber and G. Claire Ball, "Performance Evaluation of Concrete Tie Test Section on the Santa Fe at Streator," PCA report to the Santa Fe, October 1976.

[4]R. H. Prause, H. D. Harrison, J. C. Kennedy, and R. C. Arnlund, "An Analytical and Experimental Evaluation of Cross Tie and Fastener Loads in Tie Ballast Track," Federal Railroad Administration (OR&D) Report, to be published.

[5] R. H. Prause and A. Kish, "Concrete Tie Service Loads," submitted for publication to Railway Track & Structures.

[6] R. M. Bailey, "Address to the Workshop on Concrete Ties," April 12, 1977, Portland Cement Association Workshop on Concrete Ties, Chicago, Illinois.

[7] R. H. Prause, "Concrete Ties vs. Wood Tie Track Performance," compiled for the Transportation System Center under DOT-TSC-1044, Cambridge, Mass, March 18, 1977.

Chapter 7

[1] E. T. Selig and C. W. Adegote, "Observed Sub-base and Subgrade Conditions at the FAST Track," unpublished report to Federal Railroad Administration, February 1977.

[2] The fact that the gradation changes occurred under the outer rail on the curve is not necessarily significant since it was observed that even in tangent track, smaller particles appear to occur under one rail, usually the outer rail. Further investigation concerning the migration of fines and ballast construction activities are in progress.

[3] Curves shown in figure 7-6 are sight-fitted and not computed.

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