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PARAMETRIC STUDY OF TRACK RESPONSE

December 1977
Interim Report

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16. Abstract

Abstract

This report was prepared as part of the Improved Track Structures Research Program managed by the Transportation Systems Center. This program is sponsored by the Office of Rail Safety Research, Improved Track Structures Research Division, of the Federal Railroad Administration. The report presents results from a parametric study of track response using a comprehensive track analysis model. Track response parameters include rail and tie bending moments, rail displacement, tie rail seat load, and the distribution of stresses in the ballast and subgrade. The effects of variations in tie size, tie spacing, ballast depth and rail fastener stiffness are presented in graphs suitable for track design trade-off studies. Alternative wood and concrete tie track configurations are evaluated using equivalent maintenance criteria.

Maintenance criteria for track surface deterioration were reviewed to select ballast and subgrade stress parameters which can be related to track settlement rate for service loads. Formats for cumulative settlement criteria are evaluated using available laboratory data for the behavior of ballast and subgrade materials under repeated loads. Track lateral strength requirements for wood and concrete tie track are also discussed.

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PREFACE

This report was prepared by Battelle's Columbus Laboratories (BCL) under Contract DOT-TSC-1044 as part of the Improved Track Structures Research Program managed by the Department of Transportation, Transportation Systems Center (TSC). This program is sponsored by the Office of Rail Safety Research, Improved Track Structures Research Division, of the Federal Railroad Administration, Washington, D.C.

The overall objective of this program is to improve the safety and serviceability of cross tie track. Work on this contract, which is part of the Improved Track Structures Program, includes an evaluation of the technical and economic feasibility of using synthetic cross ties and rail fastener assemblies to obtain improved component life and long-term performance and a parametric study of track response. The parametric study includes a unified assessment of the effect of variations in tie size and spacing and ballast depth on track response. These are the principal variables in track design.

This is the third interim report for this contract. The first interim report was a planning document for a track measurement project. The second interim report covered the review and selection of track analysis models for predicting track response and included a statistical description of concrete tie track loads from measurements made on the Florida East Coast Railway.

Dr. Andrew Kish and Mr. Donald McConnell of the Transportation Systems Center were the technical monitor and alternate technical monitor, respectively, for the work reported herein. Their cooperation and suggestions are gratefully acknowledged. Mr. Robert Arnlund of Bechtel, Incorporated, a subcontractor on this contract, also deserves recognition for his suggestions regarding criteria for track performance measures.

METRIC CONVERSION FACTORS

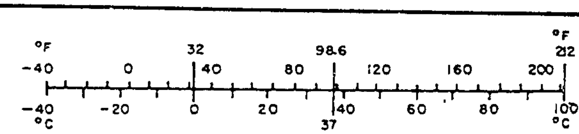
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliter	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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1. INTRODUCTION

Experiences from several foreign countries indicating advantages of longer tie life and reduced track maintenance for concrete versus wood ties have aroused considerable interest in developing concrete ties for main-line use in North America. However, little quantitative data are available for comparing wood and concrete tie loads and roadbed stresses, or long-term performance, as a basis for evaluating the technical and economic feasibility of alternative track and tie designs.

Previous work on this project resulted in the identification of the major modes of track degradation and the critical load parameters. Analysis models were selected, and developed when necessary, to provide analytical procedures for predicting track response relative to these different modes of degradation. Measurements of track loads on several sections of concrete ties on the Florida East Coast Railway were used to validate the analysis model for vertical loads.

The principal objective of the research discussed in this report was to use the track analysis model for vertical loads to develop track design guidelines which include the effects of various tie/fastener characteristics, tie spacing and ballast depth on track response. Available information relating track response to vertical settlement and deterioration of track surface geometry (profile and cross level) were reviewed to identify the critical response parameters and to select suitable performance indices. Alternative wood and concrete tie track configurations based on equivalent maintenance criteria were evaluated for use in the life cycle cost analysis planned for a later phase of this project. This cost analysis will include the effect of track maintenance frequencies, rail life and tie life on the justifiable purchase price for concrete tie track configurations that are expected to have the same life cycle cost as a standard wood tie track.

2. SUMMARY OF RESULTS AND CONCLUSIONS

A review of the current state-of-the-art on track surface deterioration indicates that there are no proven quantitative performance measures to predict the deterioration rate from a description of the railroad traffic and the track design. However, some laboratory investigations of the behavior of ballast and subgrade materials under repeated load show that the difference in the maximum and minimum principal stress, or deviator stress, is the most important parameter for determining track settlement rate. Settlement rate appears to be proportional to deviator stress raised to a power greater than one, and the settlement increases proportional to log N, where N is the number of cycles (axles). However, a quantitative cumulative settlement law which would combine the deformation in the ballast and subgrade for the variable amplitude axle loading for typical railroad traffic is not currently available.

A track analysis model, MULTA (MUlti-Layered Track Analysis), having a multi-layer representation of the track roadbed was developed in order to predict realistic stress distributions in the ballast and subgrade. This model also includes, in a unified manner, the effect of tie bending and changes in ballast depth, ballast and subgrade material properties, tie size, and tie spacing. This is a significant improvement over conventional track design practice.

Results from the MULTA model with vertical loading, show the influence of tie bending stiffness on the variations in tie/ballast pressure along the tie length. Wood ties, and small concrete ties which are flexible relative to the roadbed, cause maximum pressures under the rail seat region. Large concrete ties having a high bending stiffness relative to the roadbed stiffness can cause maximum pressure close to the tie ends. The maximum stress levels in the ballast and subgrade are major factors in track settlement, but non-uniform stress distribution on the subgrade under the tie and along the track can cause local depressions, or "rutting". These depressions will collect water resulting in possible slow drainage. The local reduction in subgrade strength from excess moisture will cause a rapid increase in settlement and pumping. For this reason, both the maximum ballast and subgrade stresses and a ratio of maximum to minimum stress as a measure of stress variation are recommended as critical factors for track design. Increasing tie spacing causes a relatively large increase in pressure ratio along the track, but the pressure variations under ties are higher and are therefore the more critical design problem. Track design data generated with the MULTA program can be used to evaluate the effects of changing ballast depth, tie size, and tie spacing on roadbed stresses. The parametric study showed that a track system with various combinations of synthetic tie size, tie spacing and ballast depth gave equal or superior roadbed stress conditions when compared to a track structure with wood ties.

Results from the parametric evaluation of vertical rail fastener stiffness showed that the distribution of track loads can be improved by using a flexible fastener with a vertical stiffness less than about 500,000 pounds per inch. Other studies show that a flexible fastener can also reduce impact loads from wheel flats and rail joints and thereby compensate for the normal increased stiffness of concrete over wood tie track. European and Russian fastener development efforts have been concentrated on designing more flexible rail fasteners. This trend has been largely ignored in the U.S., where all fasteners currently used with concrete ties are rigid relative to the track. Maintaining adequate lateral restraint against gage spread and rail rollover is the major design problem in developing a fastener with a lower vertical stiffness. However, the reduction in impact loads and the improved load distribution which can be obtained with more flexible fasteners should be adequate to encourage additional development efforts by the industry.

Track lateral strength is an important factor in maintaining track alinement under continuous traffic and for the safety aspects of train derailments. The lateral strength of unloaded track (no trains) must be sufficient to prevent track shifting from rail thermal loads. Occupied track must have sufficient lateral strength to resist both thermal loads and the lateral component of wheel loads. These lateral strength requirements can be used to determine maximum tie spacing and minimum ballast shoulder requirements for either consolidated or recently maintained track. Available data show that track maintenance operations reduce track lateral resistance to about 40% of the resistance of a well-consolidated track. Also, concrete tie track with 24-inch tie spacing shows about a 16% advantage in lateral resistance over wood tie track with 20-inch tie spacing.

The results from the parametric study of track response can be used for track design trade-off studies where the effect of tie size, ballast depth and tie spacing are of interest. Predictions of maximum subgrade deviator stress and the distribution of subgrade deviator stress under ties and along the track, are provided as a measure of relative settlement rate. The predictions of tie rail seat loads and tie bending moments for the different track configurations will be used to evaluate tie and fastener performance specifications in a later phase of this project. Wood and concrete tie track configurations expected to have equal maintenance intervals for surfacing have also been selected as a basis for subsequent life cycle cost comparisons.

3. TRACK DESIGN PARAMETRIC STUDY

Previous work [3-1] on this research program showed that the evaluation of track performance and design for vertical loads requires a capability for predicting realistic pressure distributions at the tie/ballast interface and at the ballast/subgrade interface. This requires a track analysis model which includes the effect of tie bending and in a unified manner the changes in ballast depth, ballast subgrade material properties, tie size, and tie spacing. Changes in track design which affect track stiffness (modulus), and the resulting redistribution of loads from the rail to individual ties would then be readily apparent.

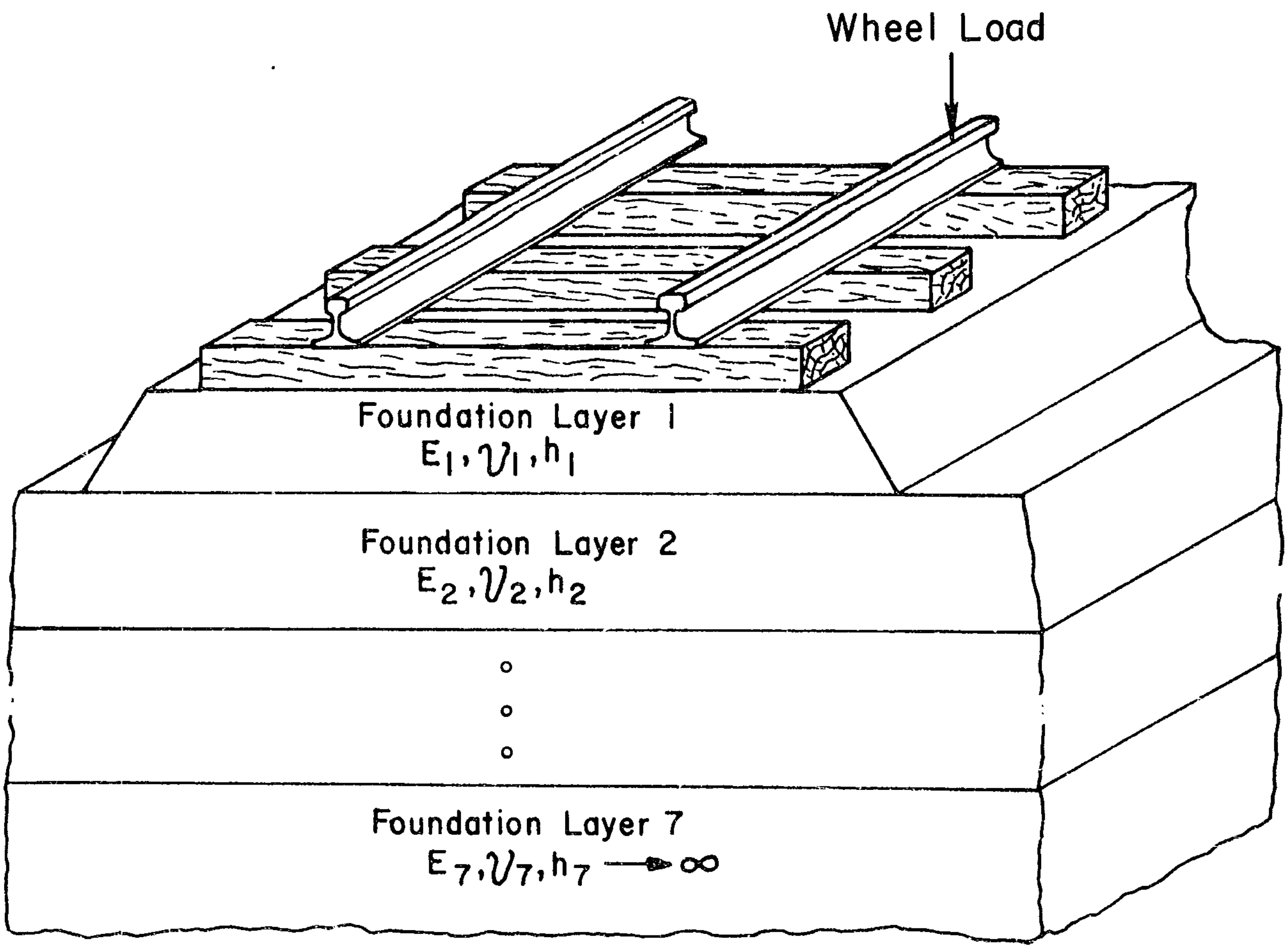
3.1 DESCRIPTION OF TRACK ANALYSIS MODEL

The analysis model selected for this program is a combination of an available multi-layer model to assess the ballast and subgrade, and a finite element model to combine the loads for individual ties and rails (load combination program). The load combination program, developed by the Association of American Railroads (AAR), was modified by BCL to incorporate influence coefficients from the multi-layer roadbed model in order to provide a complete track model.

Figure 3-1 shows a schematic of this combination model, MULTA (MUlti-Layer Track Analysis). The model provides a linear track analysis which includes single or multiple wheel loads on 2 rails supported by ties of variable size and spacing and a finite bending rigidity. The tie bearing area is divided into segments of approximately square dimensions. These are used to generate influence coefficients for pressures and displacements from the multi-layer roadbed model. Compatability and equilibrium equations form a system of equations which is solved using matrix analysis techniques to calculate ballast and subgrade stresses and rail and tie displacements.

Some important features of MULTA are:

(a) The roadbed can be modeled using 2 to 7 layers of homogeneous, isotropic elastic material, each having distinct material properties and depths. The last layer has infinite depth to represent real track construction.



E = Young's modulus

ν = Poisson's ratio

h = Layer thickness

FIGURE 3-1. SCHEMATIC FOR TRACK MODEL

(b) The stiffness of a vertical spring between each rail and tie can be adjusted to evaluate the effect of a flexible rail fastener and tie pad.

(c) The rails can be loaded by vertical loads located over a tie or between ties.

(d) The direct incorporation of tie deformation due to bending and its effect on the tie/ballast contact area eliminate the need for any assumptions regarding the "effective" contact area for a particular tie. This has been an important limitation for several previous track design procedures.

See [3-9] for a description of program use and documentation.

3.2 REFERENCE TRACK PARAMETERS

This parametric study used a MULTA model of a track section having 11 ties and separate layers for the ballast and subgrade. The track was loaded vertically at the center tie using the load for a single axle of a freight car having a gross weight of 240 kips and a wheel load of 30 kips. Since this is a linear analysis program, results for heavier or lighter wheel loads can be obtained by direct scaling. MULTA presently handles a single vertical load per rail for each computer run, so adjacent axle loads were not simulated. However, adjacent axle loads could be included by superposition. This effect is discussed later in the report. The parameters of particular interest for this study were:

(a) Rail and tie displacement
(b) Vertical rail seat load
(c) Tie bending moments at the center and rail seat regions
(d) Maximum rail bending moment
(e) Displacement (strain) throughout the foundation
(f) Bulk stress at selected points in the foundation
(g) Vertical and deviatoric stress at selected points throughout the foundation
(h) Tie/ballast interface pressures.

Table 3-1 lists the track parameters which were included in this study. Average values of tie bending stiffness are listed in Table 3-1 since MULTA

TABLE 3-1. TRACK MODEL PARAMETERS

Rail: 136 lb/yd, $I = 94.9$ in.^{4}, $E = 28.9 \times 10^6$ psi, $A = 13.35$ in.^{2}

Wood Tie: 7 in. thick, 9 in. wide, 102 in. long, $EI = 470 \times 10^6$ lb-in.^{2}
spacing = 19.5 in.

Concrete Tie: 102 in. long spacing = 20, 24, 30 in.

a. Small: 10 in. wide, $EI = 764 \times 10^6$ lb in.^{2}, $A = 60$ in.^{2}
b. Medium: 10.5 in. wide, $EI = 1011 \times 10^6$ lb-in.^{2}, $A = 64$ in.^{2}
c. Large: 10.5 in. wide, $EI = 2341 \times 10^6$ lb-in.^{2}, $A = 82$ in.^{2}

Ballast/Subballast: $E_1 = 30$ ksi, $\nu_1 = 0.4$, depth = 12, 24, 36 in.

Subgrade: $E_2 = 10$ ksi, $\nu_2 = 0.4$, depth = infinite

Rail Fastener Stiffness: 1, 2, 4, 10, 40 $\times 10^5$ lb/in.

Wheel Load: 30 kips

Note: Reference track parameters are underlined.

assumes the tie to have constant cross sectional area. The value of EI listed for the small concrete tie is the average value for the tie that is used on the Florida East Coast Railway. The other values of EI listed in Table 3-1 were similarly derived from data for concrete ties which meet current specifications of the American Railway Engineering Association (AREA). Ballast modulus (E_1) and subgrade modulus (E_2) values were picked to represent typical track foundation properties. Variations in tie size, stiffness, and spacing and ballast depth were selected as the key parameters. The effect of varying rail size can be evaluated adequately using conventional track design procedures based on beam-on-elastic-foundation (BOEF) equations when the track modulus has been established. Work by Tayabji and Thompson [3-2] shows that variations in ballast and subgrade modulus over a typical range for field conditions do not cause large changes in predicted ballast and subgrade stresses. Track degradation under repeated load would, however, vary considerably for different materials. The roadbed material properties used for this study are based on average values reported in [3-2]. For purposes of the parametric study, a reference track designated by the underlined parameters in Table 3-1 was used for base line comparisons.

3.3 TRACK MODEL EVALUATION

The MULTA model was evaluated previously [3-1] by comparing predicted and measured data for the distribution of tie/ballast pressures along the tie length and by comparing measured and predicted tie bending moments. It was necessary to select values of Young's modulus for the ballast and subgrade to match the measured track modulus by comparing data for average tie plate loads. When this was done, the predicted and measured pressures and bending moments were in good agreement except for ties which have a severe centerbinding condition. This nonuniform support condition cannot be simulated with MULTA because a uniform elastic support model is used for the roadbed. However, even ties which are centerbound for light cars exhibit a more uniform support condition when loaded by heavy cars, so MULTA predictions are useful.

A second evaluation of the program was made by comparing results from Thompson and Tayabji [3-2] using the ILLI-TRACK model. This is a two-dimensional finite-element code developed at the University of Illinois. The track structure is analyzed in two stages. The first stage analyzes the track in the longitudinal direction along the rail. These results are used as input to the second stage which analyzes the track structure in the plane of a tie. Although the elements in the analysis are two-dimensional, a psuedo third dimension is generated by having the element width increase with depth to account for stress dissipation through the foundation.

The assumption of element width and an initial "effective tie bearing length" are critical steps in this analysis procedure. The rule of thumb for choosing effective tie-bearing length has been to use one-third of the tie length under each rail seat [3-1]. However, the ILLI-TRACK model [3-2, 3-3] uses an effective bearing length under each rail of 18 inches, which is less than one-fifth the tie length.

A comparison of predicted track response for similar wood tie track parameters is shown in Table 3-2. It is important to realize that the ILLI-TRACK model uses stress dependent ballast and subgrade modulus values and the roadbed finite elements can include a failure criterion. The ballast and subgrade modulus values in MULTA are not stress-dependent. However, the gross differences shown in the comparison are due to the extremely high pressures on the ballast at the tie/ballast interface resulting from the use of a very small effective tie bearing area in ILLI-TRACK. Increasing this bearing length would reduce the predicted pressures and also reduce the predicted rail deflection and rail bending moment. A key difference between the two models is the necessity of assuming an initial tie bearing area for ILLI-TRACK, whereas tie deformation and contact area are included directly in the MULTA model.

3.4 DISTRIBUTION OF TRACK LOADS

This section of the report shows typical distributions of ballast and subgrade pressures and tie bending moments for individual ties and the distribution of loads along the track. These results are discussed to provide a background for the summary of track response data given in the following section.

The vertical pressures at the tie/ballast interface and on the subgrade at the ballast/subgrade interface are shown in Figure 3-2 for ballast depths

TABLE 3-2. COMPARISON OF RESULTS FROM MULTA AND ILLI-TRACK MODELS

	ILLI-TRACK Model (1)		MULTA Model
	With Failure Criteria	No Failure Criteria	
Wheel Load, kips	30	30	30
Rail Size, lb/yd	136	136	136
Wood Tie Size	7"x8"x96"	7"x8"x96"	7"x9"x102"
Ballast Properties			
Modulus E, ksi	30 (2)	30 (2)	30
Poisson's Ratio, ν	0.35	0.35	0.4
Depth, in.	12	12	12
Subgrade Properties			
Modulus E, ksi	10(2)	10(2)	10
Poisson's Ratio, ν	0.47	0.47	0.4
Depth, in.	$\geq$270	$\geq$270	∞
Tie Spacing, in.	20	20	19.5
Rail Deflection, in.	0.10	0.07	0.05
Rail Bending Moment, inch-kips	297	240	195
Ballast Pressure, psi	114.3	53.6	30.8
Subgrade Pressure, psi	26.4	19.6	11.5

(1) ILLI-TRACK Results from Table 1 of [3-3].
(2) Initial modulus in stress-dependent model. MULTA uses constant modulus in each layer.

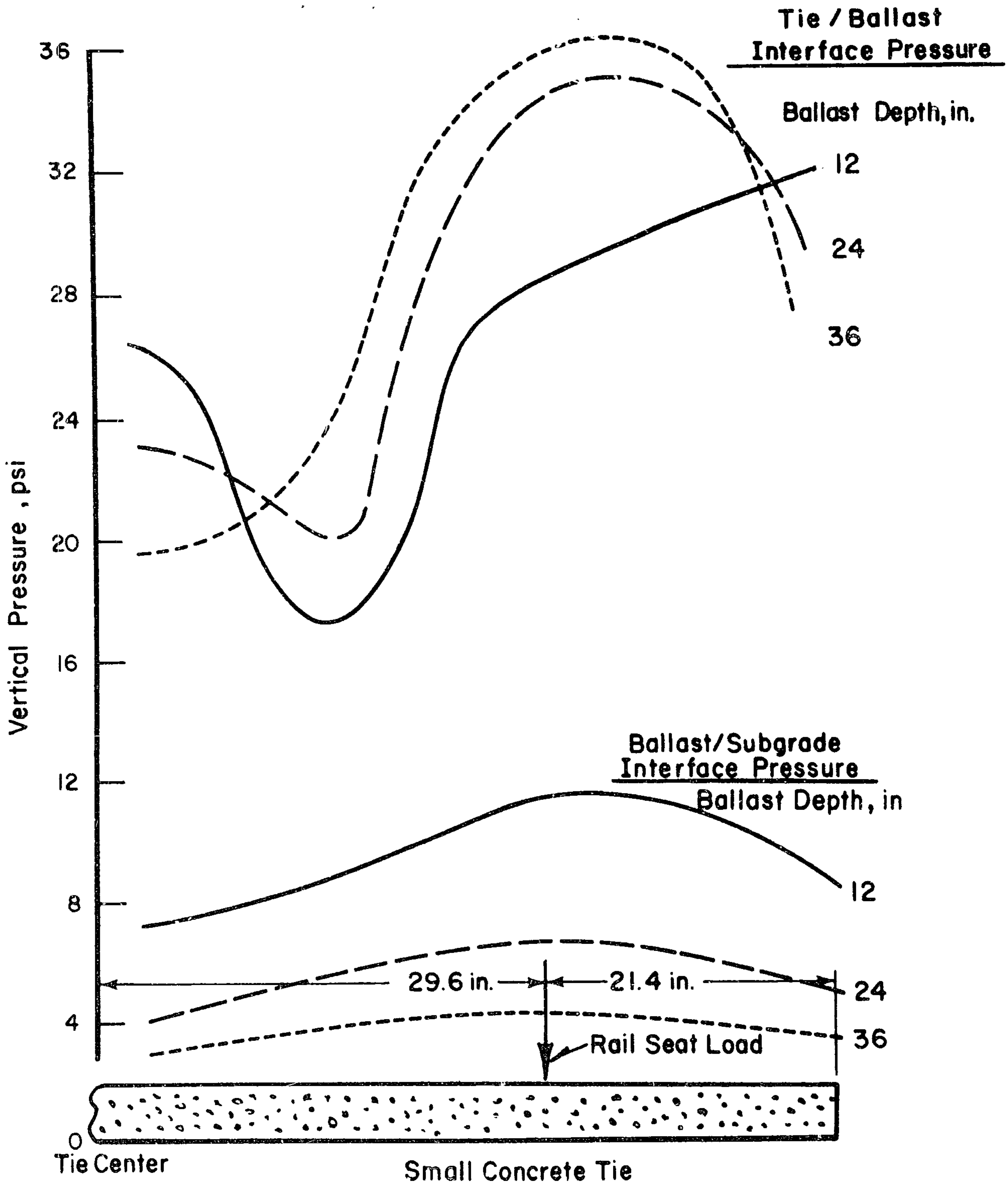


FIGURE 3-2. ROADBED VERTICAL PRESSURE DISTRIBUTION

of 12, 24, and 36 inches. The rather strange looking pressure distribution under the tie at the tie/ballast interface is caused by the high stiffness of the tie relative to the foundation. If the tie were quite stiff relative to the roadbed, the pressure distribution under the tie would resemble that for a rigid punch on an elastic medium, as shown in Figure 3-3. The vertical pressure at the ends of the tie would be very high (theoretically infinite) and reduce to some minimum value at the tie center. If the tie were flexible relative to the roadbed, the vertical pressure distribution at the tie/ballast interface will resemble that for a flexible beam on an elastic medium, as shown in Figure 3-3. The vertical pressure will be low at the tie ends and reach a maximum under the rails.

The tie/ballast pressure distributions shown in Figure 3-2 for 12- and 24-inch ballast depth are a combination of the "rigid punch" and "flexible beam" configurations. The pressure tends to increase at the end of the tie. The area of reduced pressure between the rail seat and tie center is caused by the local bending of the ballast layer in response to the high end loads. It is possible to get tensile forces for this type of loading from the model. This would not occur with real ballast materials.

As the ballast depth increases to 36 inches, the roadbed becomes stiffer relative to the tie, and the vertical pressure approaches the classical case of the flexible beam on an elastic foundation. Figure 3-2 also shows that the local variations in pressure at the tie/ballast interface are attenuated in the ballast layer. The vertical stress distribution approaches the classical pressure distribution at the subgrade. This classical response was always achieved for ballast depths greater than 12 inches.

Figure 3-4 shows the deviatoric (σ_D) and bulk (θ) stress distributions along the tie at selected depths through the foundation. Thompson and Knutson [3-4] have shown the dependence of resilient modulus on these quantities. For ballast material, the resilient modulus, E_R, is a function of θ, while for typical subgrade materials, the resilient modulus is a function of σ_D.

Figure 3-4 shows θ midway through the depth of the ballast to give a psuedo-average value of bulk stress in the ballast. The deviatoric stress is shown midway through the ballast depth and at the ballast/subgrade interface. Deviatoric stress is monitored at these two locations because Raymond's work [3-5] shows that foundation material failure is a function of deviatoric stress

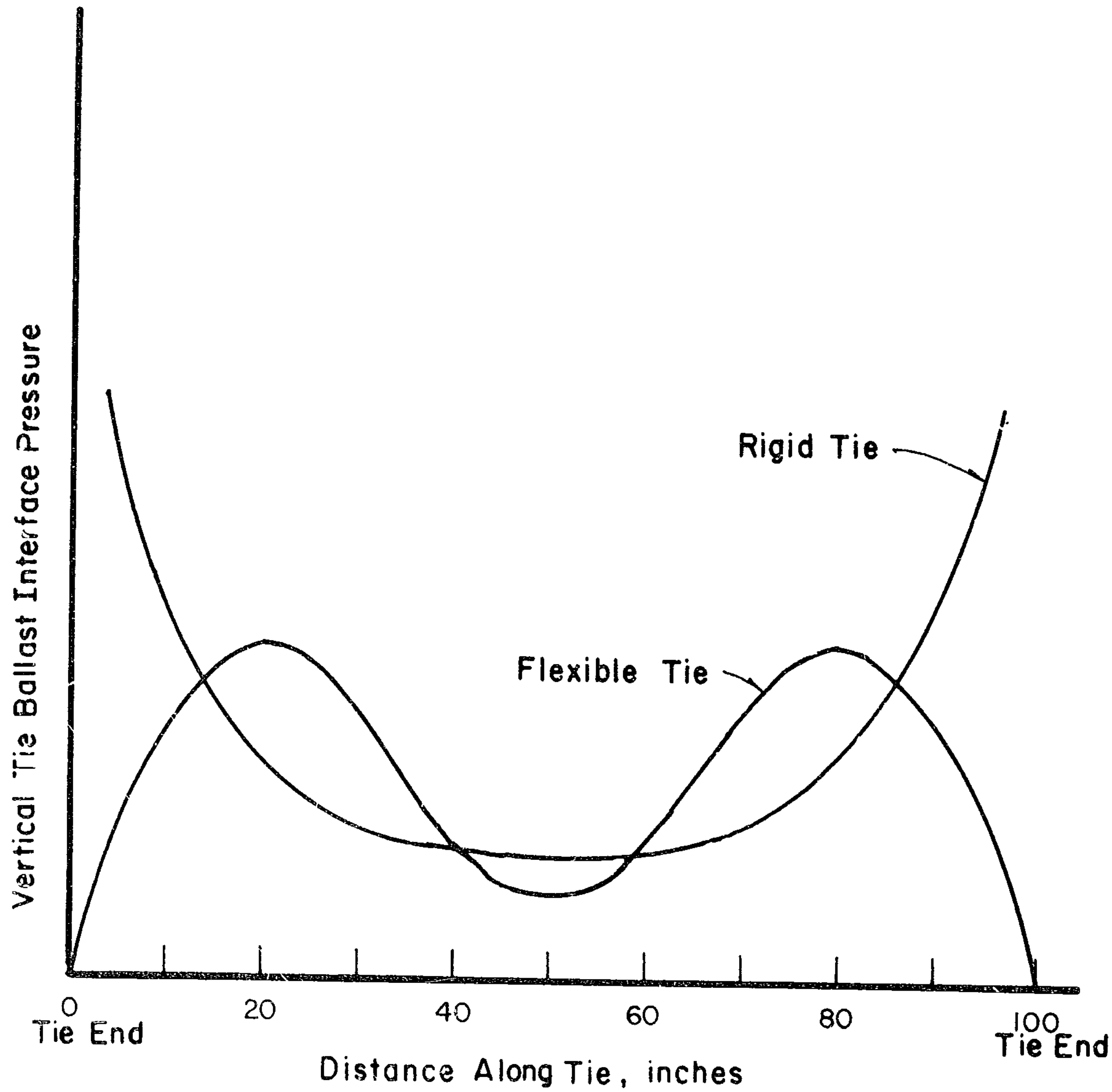


FIGURE 3-3. EFFECT OF TIE STIFFNESS ON ROADBED PRESSURE DISTRIBUTION

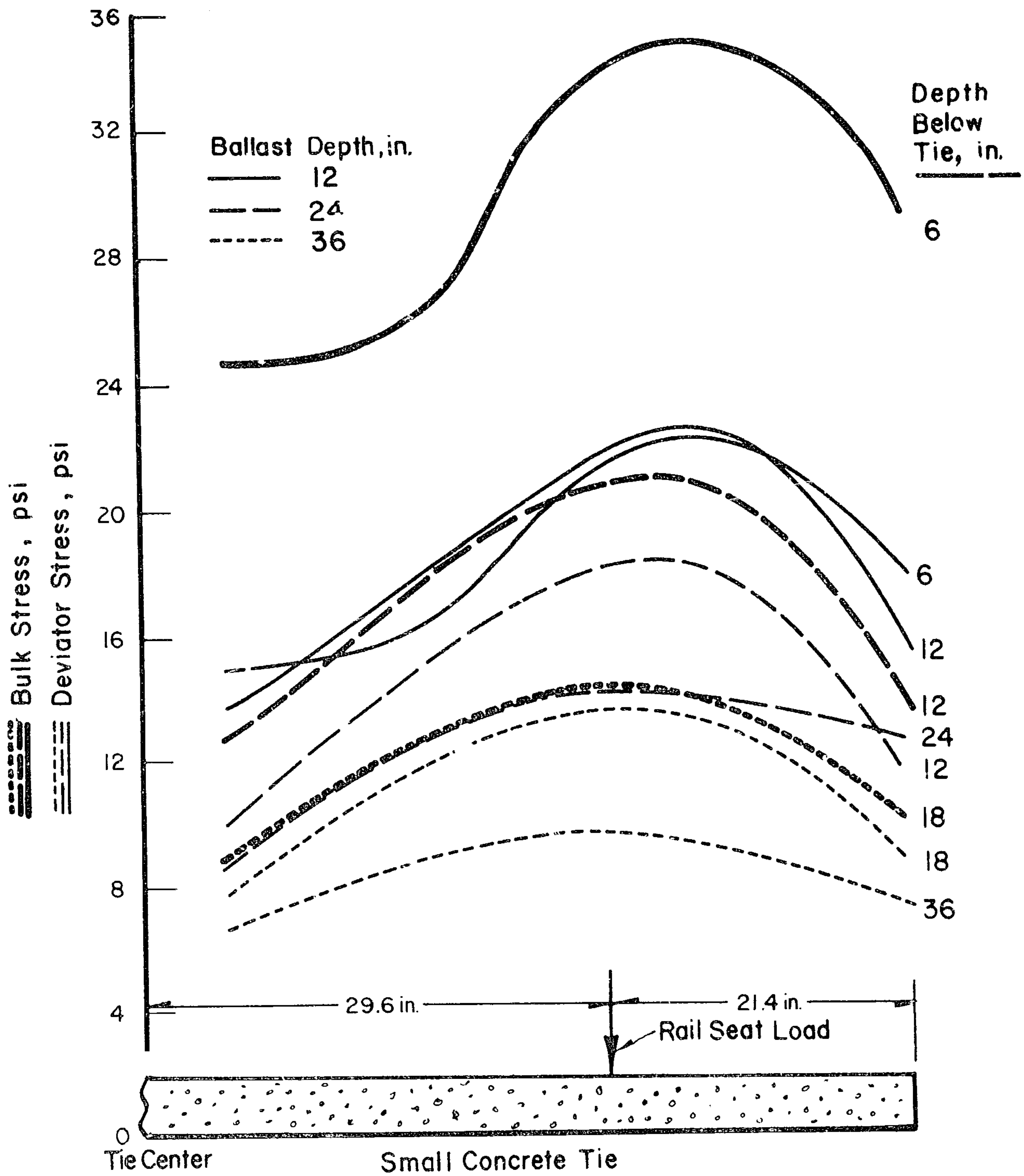


FIGURE 3-4. DEVIATORIC AND BULK STRESS DISTRIBUTIONS

for both ballast and subgrade materials. Therefore, θ and especially σ_D are used to evaluate degradation in the ballast and subgrade. The reduction of deviatoric stress by judicious selection of track design parameters is one of the main interests of this parametric study.

Unlike the vertical pressure distribution in Figure 3-2, both deviatoric and bulk stress in Figure 3-4 have a classical distribution along the tie length that is comparatively independent of the relative tie and foundation stiffnesses. Similar to the vertical pressure, σ_D and θ reduce in magnitude through the depth of the roadbed. Summary results which show the influence of tradeoffs of tie size, spacing and ballast depth on σ_D and θ will be presented and discussed in a later section.

Tie bending moment distributions are shown in Figure 3-5 for the small concrete tie. For each of the ballast depths investigated, the tie bending moment reaches a maximum positive value at the rail seat region and a maximum negative value at the tie center. Particular attention is paid to the manner in which the magnitude of the tie bending moments dissipates as distance from the loaded tie increases. For all practical purposes, the tie bending moments are negligible beyond two ties adjacent to the loaded tie. Therefore, the effect of adjacent axles located 70 inches away can be neglected for this relatively stiff track. Also, the maximum bending moments are insensitive to changes in track stiffness obtained by increasing ballast depth. It is important to remember that the predicted tie bending moments shown in this report correspond to the pressure distributions shown in Figure 3-2 for a uniform roadbed. They do not represent a centerbound condition or other types of non-uniform support, such as voids under the rail seat region that may occur in service.

Figure 3-6 shows the displacement profile of the track system as a function of ballast depth. Increasing ballast depth increases the track stiffeness and reduces rail displacement. The displacement profile is interesting because it is quite different than that predicted by the classical beam-on-elastic-foundation analysis. The displacement influence length in Figure 3-6 is much greater than the influence length for tie loads shown in Figure 3-7, which indicates that the influence of load transfer in the foundation has a pronounced effect. This is neglected in beam-on-elastic-foundation analyses. Regarding the influence of adjacent axle loads on displacement, it appears that the maximum displacement of the loaded tie would be increased by about 25%. This is not negligible, but far from a dominant effect.

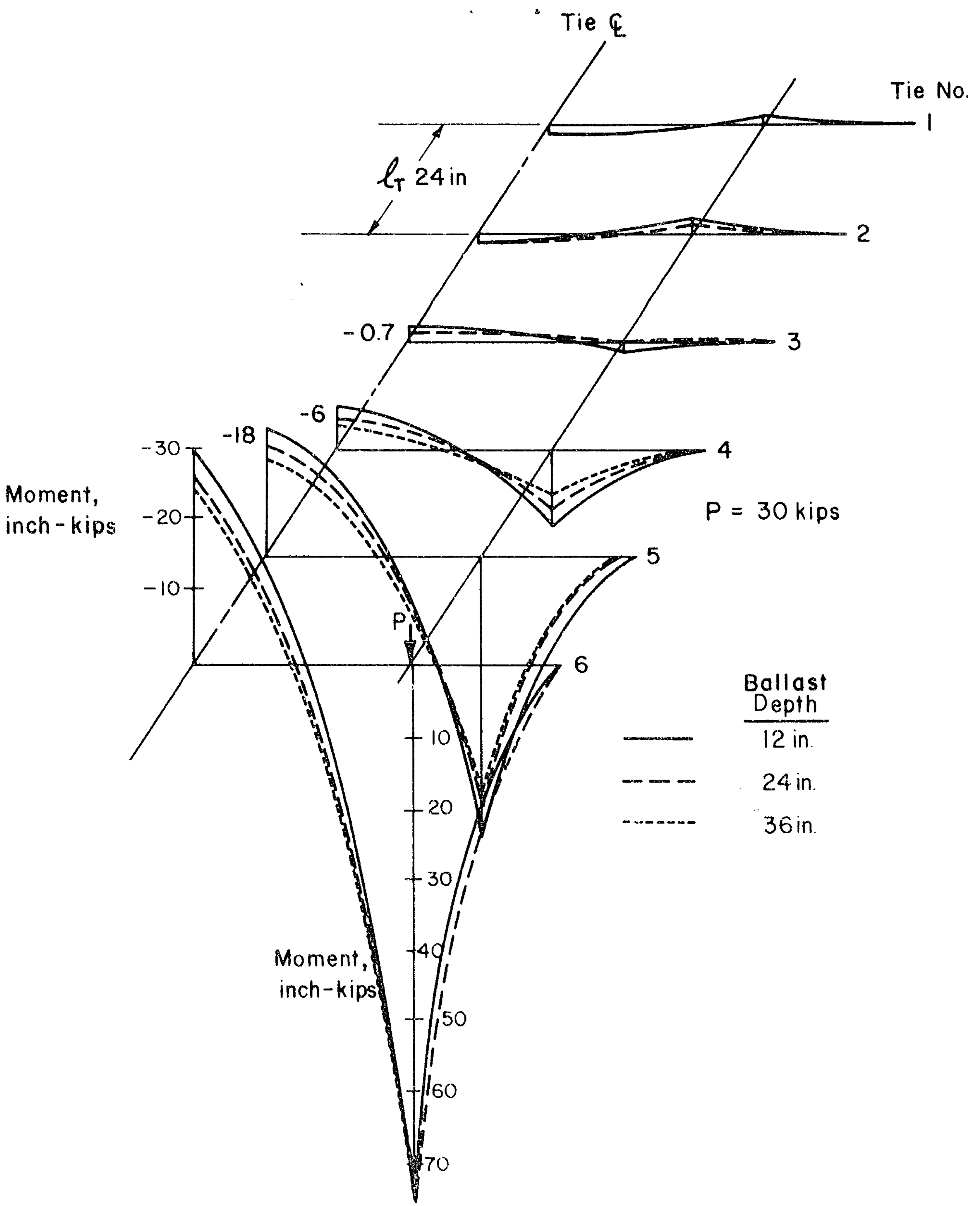


FIGURE 3-5. TIE BENDING MOMENT DISTRIBUTION

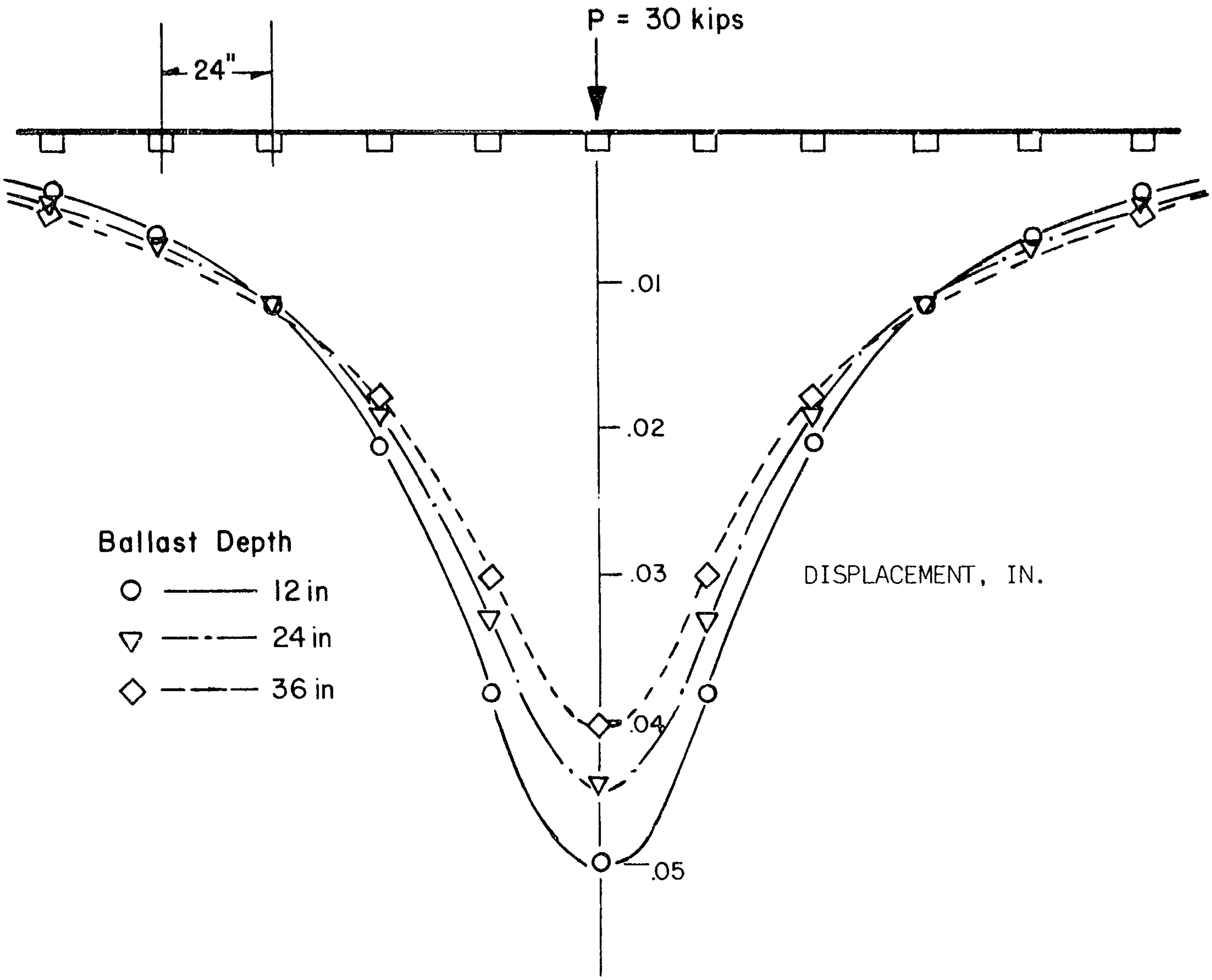


FIGURE 3-6. RAIL DISPLACEMENT PROFILE

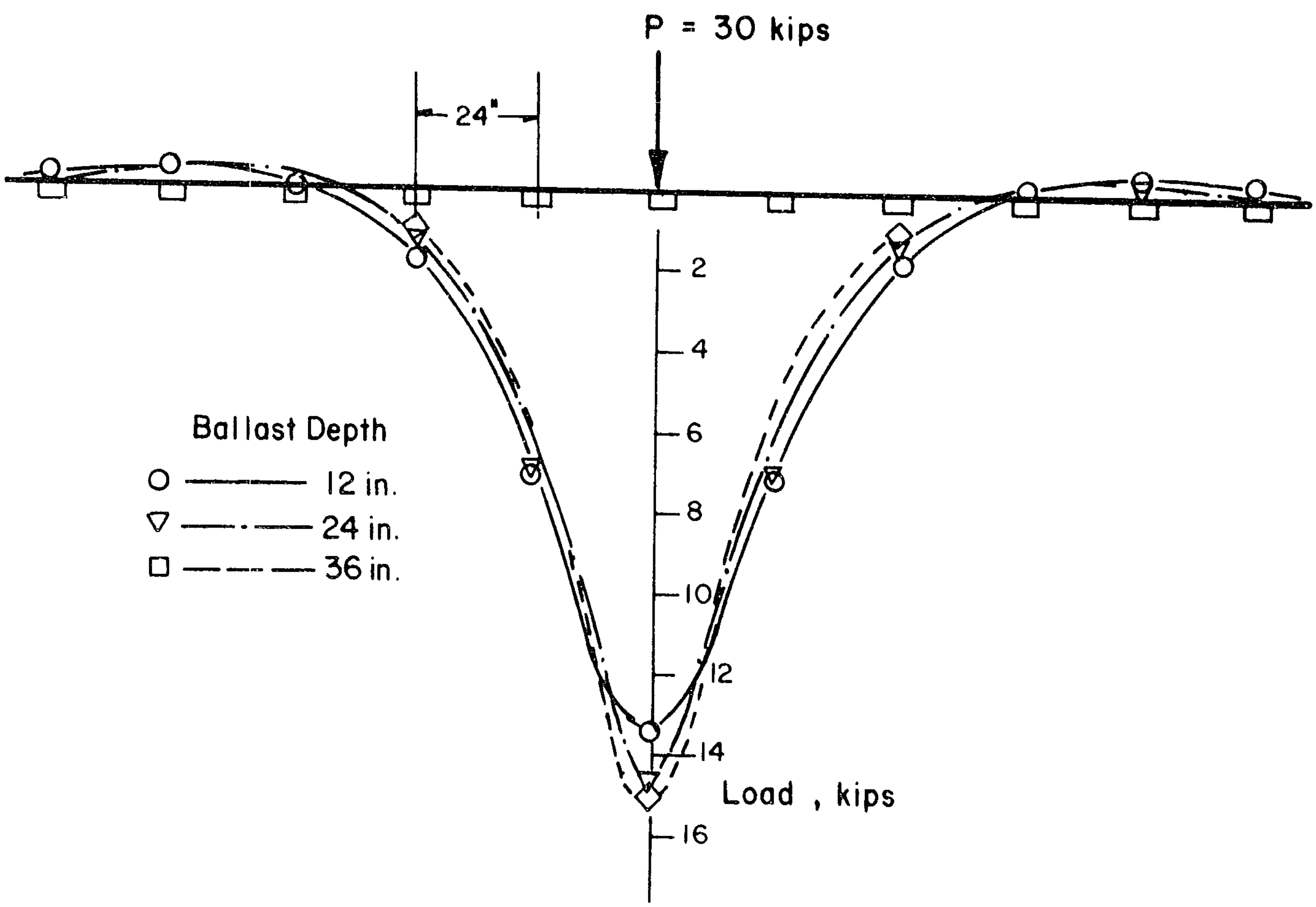


FIGURE 3-7. RAIL SEAT VERTICAL LOAD

The rail seat vertical load distributions in Figure 3-7 show that increasing ballast depth (track stiffness) increases the maximum tie load and shortens the load influence length. The loaded tie receives approximately 45% of the applied axle load for a 12 in. ballast depth and about 50% of the applied axle load for a 36 in. ballast depth. The maximum rail seat load is not affected by adjacent axle loads for 70 in. axle spacing.

3.5 TRACK MODULUS

The track modulus U is defined as the force per inch of rail required to depress the track roadbed 1-in. Traditionally, this parameter has been used to quantify the effective stiffness, or resilience, of a track structure, and it is a key parameter in the beam-on-elastic-foundation analysis procedure used for conventional track design. For this reason, the MULTA results have been used to calculate an effective track modulus in order to give a recognizable measure of the roadbed stiffness.

Figure 3-8 shows the range of track modulus data included in the parametric study of tie spacing, ballast depth and tie size. Track modulus increases with increasing values of tie size and ballast depth, and decreases with an increase in tie spacing. This effect of tie spacing is consistent with conventional track design procedures. However, the effect of ballast depth on track modulus is not usually considered, and it is very significant.

The calculations of track modulus shown in Figure 3-8 are based on the beam-on-elastic-foundation equations for vertical rail seat load in the form

$$U = 4EI \left[\frac{2}{\ell_t} \left(\frac{q_o}{P} \right) \right]^{4} \qquad (3\text{-}1)$$

where

q_o = maximum rail seat load predicted by MULTA
P = wheel load
ℓ_t = tie spacing
EI = rail bending stiffness

Equation (3-1) is one of two forms that is used to calculate track modulus U. The other form is based on maximum rail displacement Y_o. Equation (3-1) is based on knowing the tie plate reaction q_o. As pointed out in [3-1], it is desirable to have an average value of q_o from several instrumented tie plates within a test site to minimize tie-to-tie variations. Both forms are derived from beam-on-elastic-foundation theory. If the track system in reality behaves as a beam-on-elastic-foundation, then either form can be used to calculate U and the answers will be identical. However, if the shear coupling in the roadbed is significant, the track does not behave according to the assumptions used for the beam-on-elastic foundation, and the results from estimates of track modulus using measured data for q_o and Y_o will not give equivalent values for U. This is also true for the MULTA model where the shear coupling is appreciable in the simulation of the roadbed.

The measurements on the Florida East Coast Railway [3-1] showed that using the average maximum rail seat load q_o to calculate U gives results that are more consistent with the loads and moments than using rail displacements. As mentioned previously, the rail seat load distribution predicted by MULTA is qualitatively similar to the results from the beam-on-elastic-foundation solution and the Florida East Coast measurements, whereas the displacement distribution is different from beam-on-elastic-foundation solution because of coupling in the roadbed. Figure 3-9 shows the data from Figure 3-8 plotted versus track modulus based on maximum rail deflection for reference purposes. The track modulus values calculated from MULTA rail displacements range from 1/2 to 1/3 those calculated from MULTA rail seat loads, and are in the range of other typical measured track modulus data for concrete tie track.

The calculations of track modulus using maximum rail displacements, Y_o, are based on the equation

$$U = \left[\frac{(P/Y_o)^4}{64\ EI}\right]^{1/3} \qquad (3-2)$$

3.6 SUMMARY OF TRACK RESPONSE

The following sections summarize the results from the parametric study of tie size, tie spacing and ballast depth in graphs suitable for track design and performance trade-off studies.

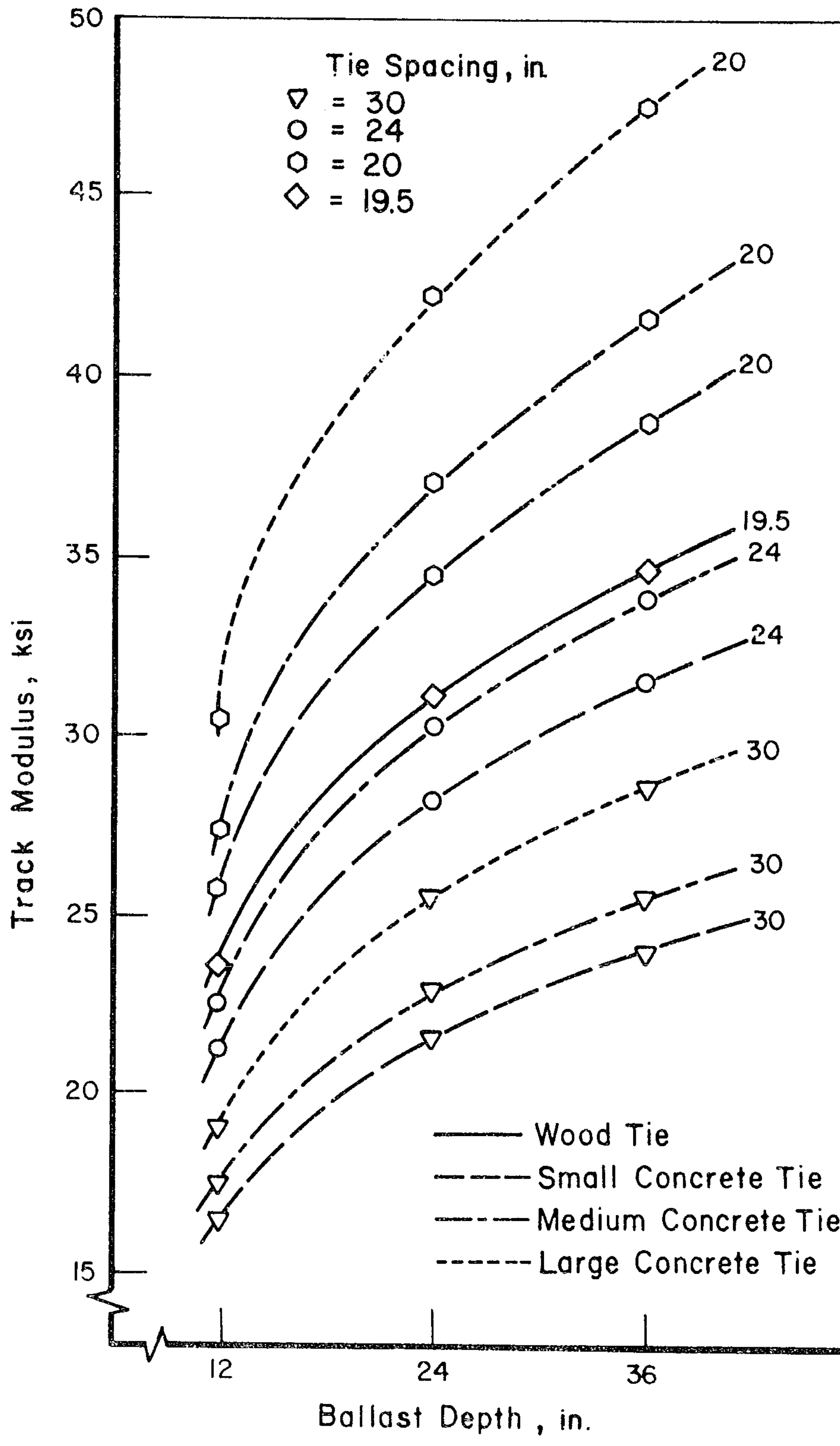


FIGURE 3-8. EFFECT OF BALLAST DEPTH, TIE SIZE AND TIE SPACING ON TRACK MODULUS CALCULATED FROM MULTA RAIL SEAT LOADS

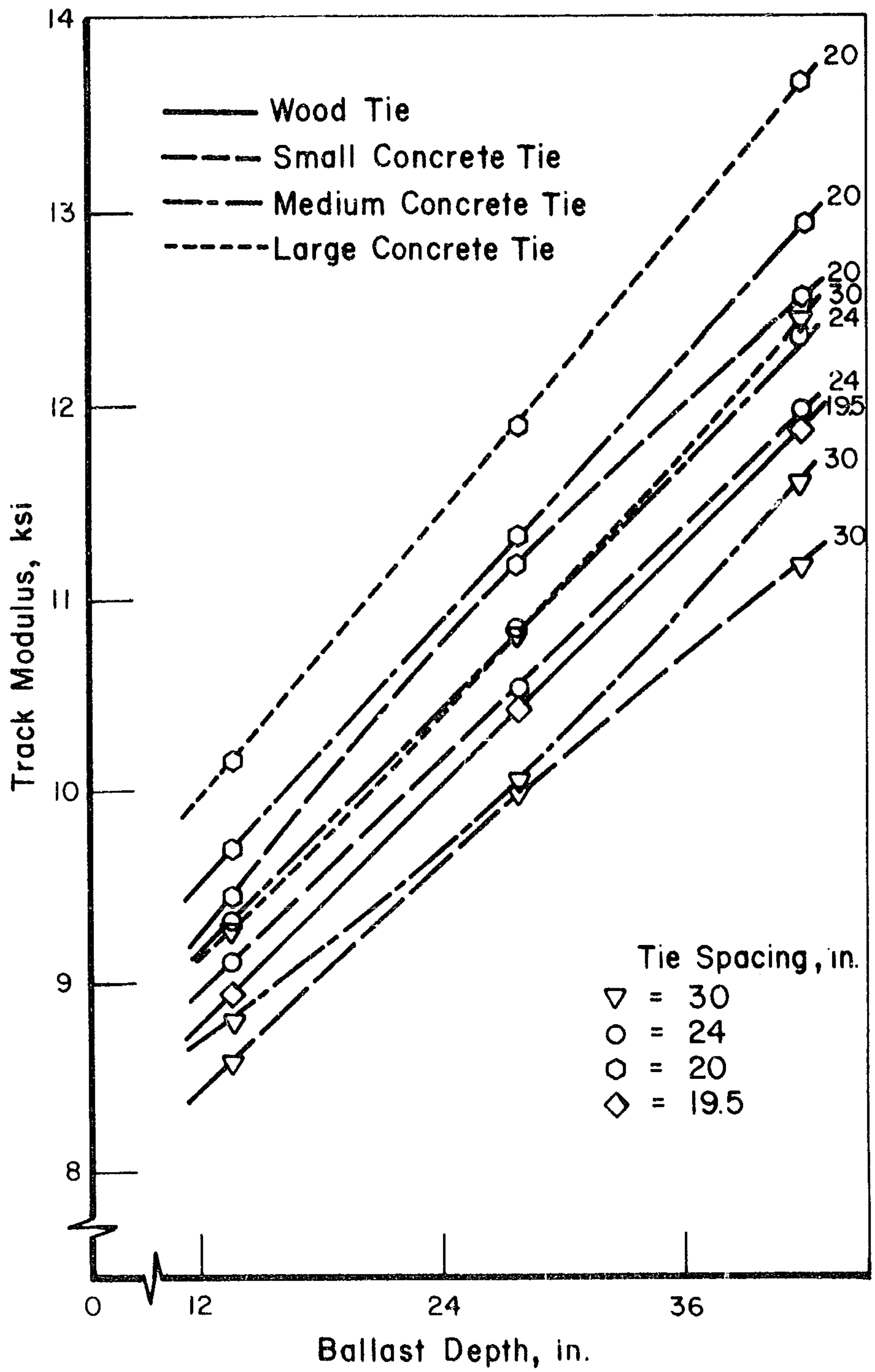


FIGURE 3-9. EFFECT OF BALLAST DEPTH, TIE SIZE AND TIE SPACING ON TRACK MODULUS CALCULATED FROM MULTA RAIL DISPLACEMENTS

3.6.1 Tie Bending Moments

Maximum tie center bending moments normalized to the wheel load are shown in Figure 3-10. The center bending moment increases as tie spacing and tie size increase, but increasing the ballast depth reduces the bending moment. Tie center moments increase approximately 40 percent going from the small concrete tie to the large concrete tie and approximately 75 percent going from the wood tie to the large concrete tie. The tie center moments decrease about 16 percent going from a 12-in. ballast depth to a 36-in. ballast depth.

Figure 3-11 shows that tie rail seat bending moment increases with tie size, tie spacing and ballast depth. Both tie center and rail seat moments increase significantly with tie size and tie spacing. Rail seat bending moments increase about 10 percent when using large concrete ties instead of small concrete ties, and about 23 percent when using large concrete ties in place of wood ties. Rail seat moment increases less than 3 percent going from a 12-in. ballast depth to a 36-in. ballast depth. Ballast depths greater than 36 inches have a negligible effect.

These predicted tie bending moments do not include the effects of non-uniform support conditions sometimes found in track. Tie center bending moments in particular can be much higher with centerbound ties and can change with end-bound ties.

3.6.2 Rail Displacement

Figure 3-12 shows rail displacement normalized by the applied wheel load as a function of ballast depth, tie size, and tie spacing. This comparison shows a slight increase in displacement with tie spacing--about 10 percent increase when tie spacing changes from 20 inches to 30 inches. This is a much smaller change than would be predicted by conventional track design procedures. Rail displacement decreases slightly (about 7 percent) when the tie size is changed from the small to the large concrete tie. Rail displacement also decreases with an increase in track stiffness, i.e., a deeper ballast. When the ballast depth is increased from 12 in. to 36 in., the displacement is reduced by about 20 percent. Figure 3-12 shows that synthetic ties of different size, spacing and ballast depth can reduce track displacements from the levels of a wood tie track structure, and this has been confirmed in practice.

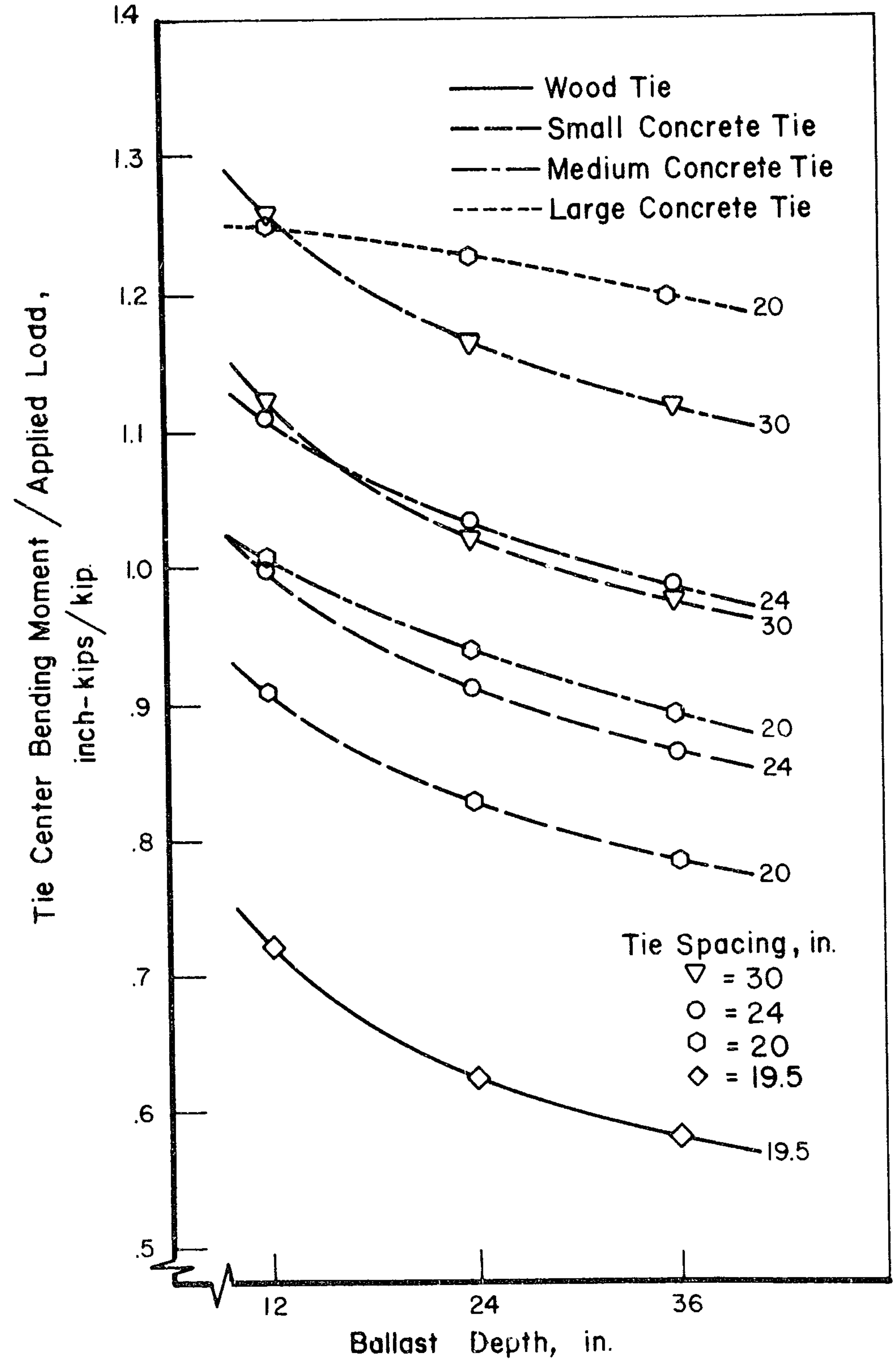


FIGURE 3-10. MAXIMUM TIE CENTER BENDING MOMENT

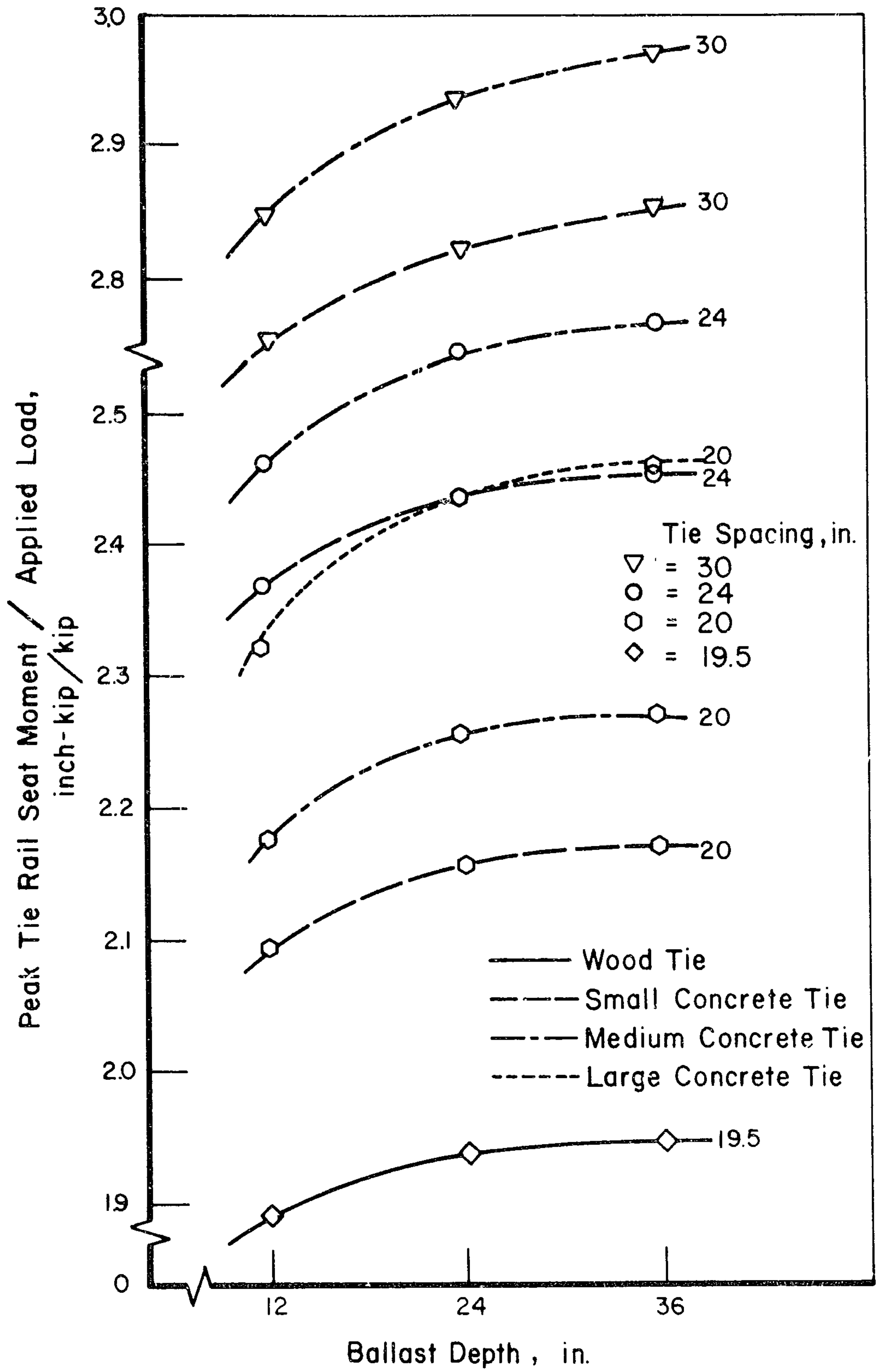


FIGURE 3-11. MAXIMUM TIE RAIL SEAT BENDING MOMENT

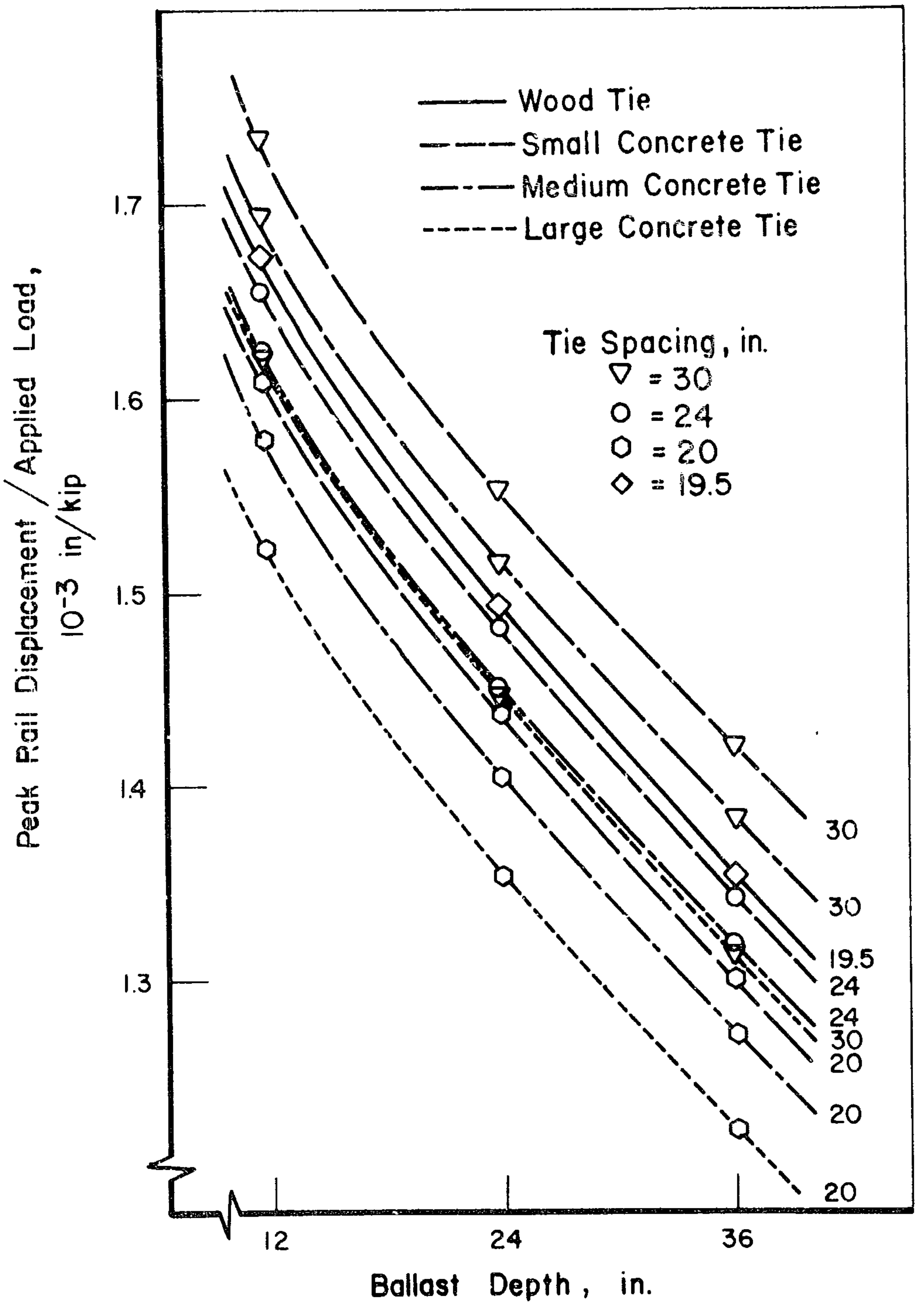


FIGURE 3-12. MAXIMUM RAIL DISPLACEMENT

3.6.3 Rail Bending Moment

Rail bending moment decreases with an increase in ballast depth, tie spacing, and tie type (size), as shown in Figure 3-13. The rail moment decreased approximately 13 percent when ballast depth was increased from 12 in. to 36 in. The effect of changing tie spacing and tie size was quite small, as expected. It is apparent that each of the synthetic tie configurations offer an improvement in rail bending compared to the wood tie track structure on the same roadbed.

3.6.4 Rail Seat Load

Figure 3-14 shows the variation in vertical rail seat load, q_o. The rail seat load increases with a corresponding increase in each of the varied parameters, as expected. When the small synthetic tie configuration was changed to the large tie configuration, q_o increased about 6 percent. Changing ballast depth from 12 in. to 36 in. increased q_o by about 13 percent. An increase in synthetic tie spacing from 20 in. to 30 in. amounted to about a 33 percent increase in q_o. It is apparent from Figure 3-14 that the rail seat loads will be consistently higher with synthetic ties used in place of wood ties because of the increased tie spacing and higher track stiffness of the wider ties.

3.6.5 Ballast Stresses

It is important to monitor deviatoric and bulk stresses because deviatoric stress is closely related to track degradation rate. Figures 3-15 and 3-16 show the stress levels in the ballast for different tie sizes, tie spacing, and ballast depth. Figure 3-15 shows that comparable levels of peak deviator stress midway through the ballast depth can be obtained for several combinations of tie spacing and ballast depth. With a 12 in. ballast depth, an increase of 1-1/2 to 3 inches is about equivalent to a 1-in. reduction in tie spacing in terms of its effect on reducing ballast pressure. Maximum deviator stress decreases rapidly as ballast depth increases--as ballast depth is increased from 12 in. to 36 in., decreasing about 44 percent for the 30-in. tie spacing, and about 36 percent for the 19.5-in. spaced wood ties, and about 30 percent for the

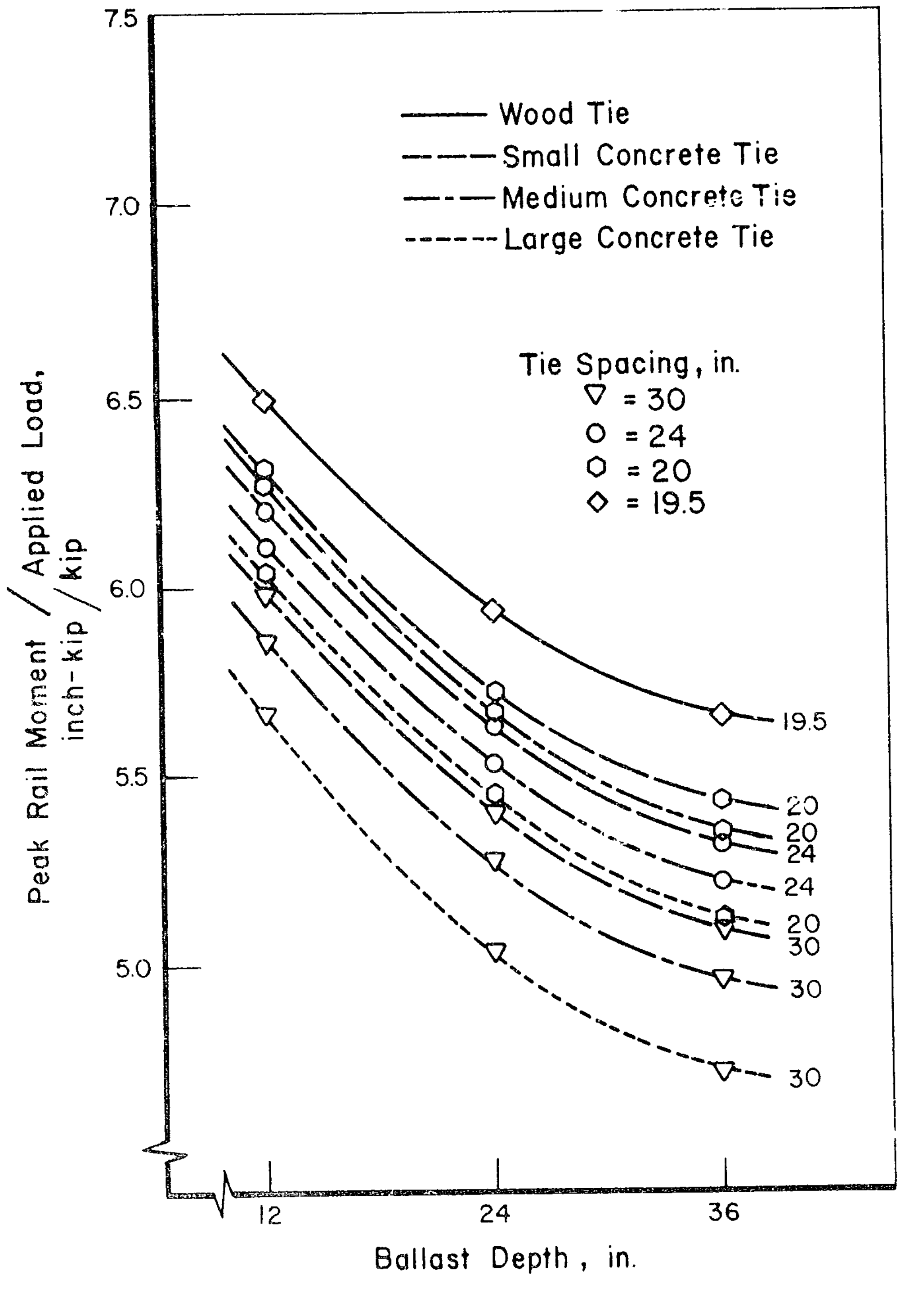


FIGURE 3-13. RAIL BENDING MOMENT

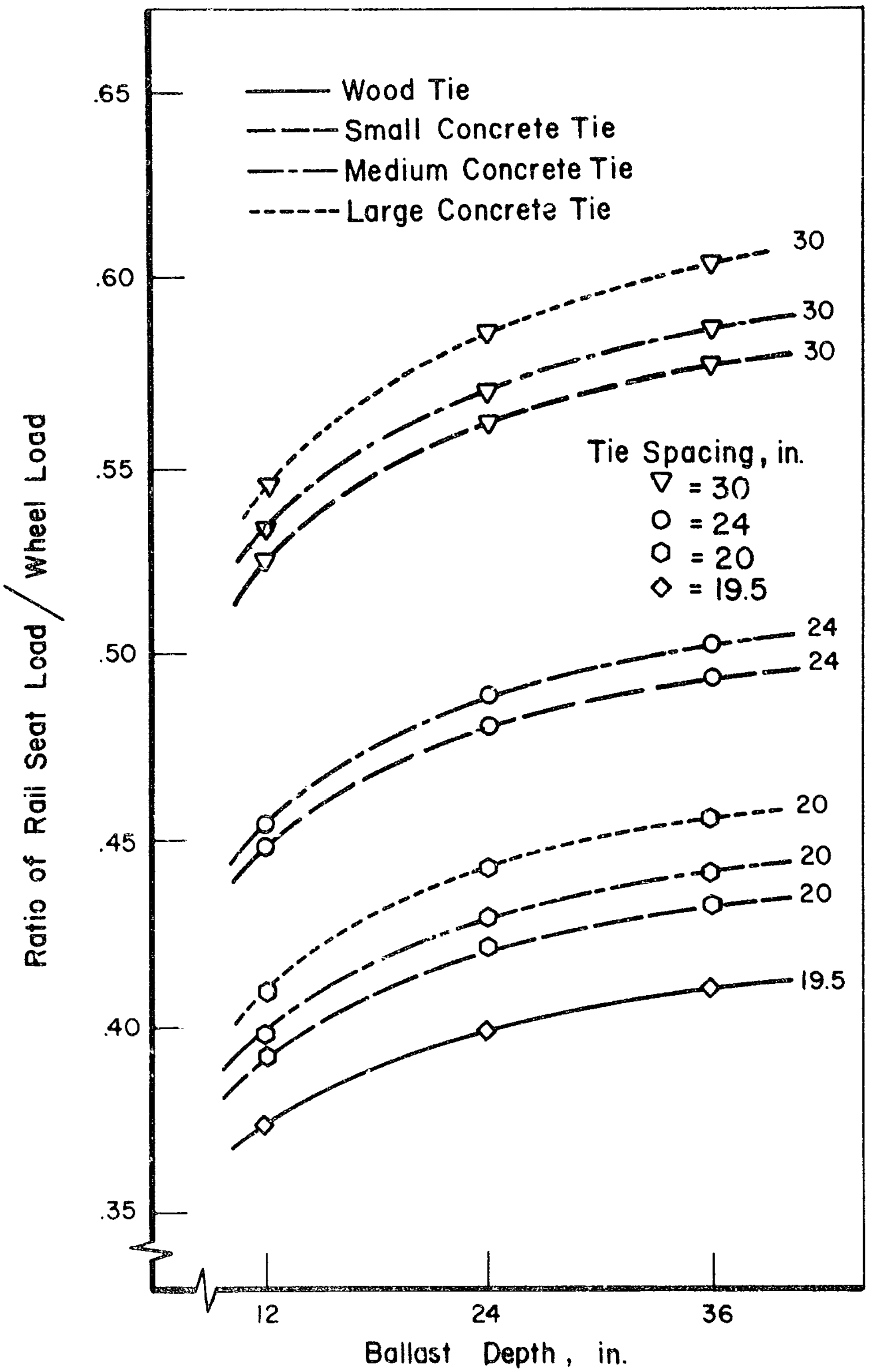


FIGURE 3-14. RAIL SEAT LOAD

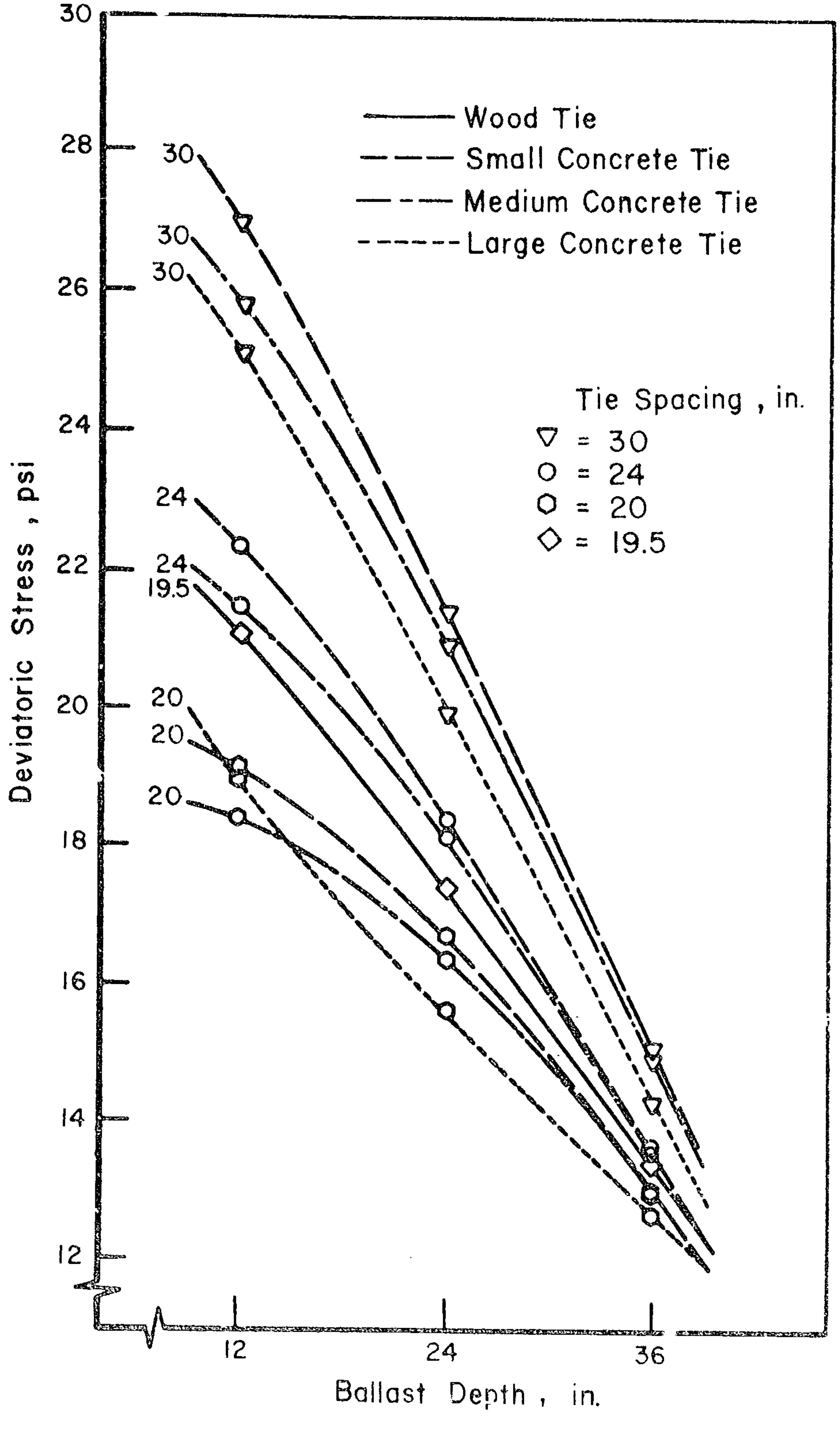


FIGURE 3-15. MAXIMUM DEVIATORIC STRESS MIDWAY THROUGH BALLAST

20-in. spaced concrete ties. As ballast depth increases, the level of deviator stress converges to within 2.5 psi of a common value for all ties and tie spacing.

Figure 3-15 also shows that increasing tie size for a given tie spacing reduces the level of deviator stress in general, but the range of tie width evaluated in this study was limited. However, an anomaly exists with 20-in. tie spacing because for thin layers of ballast the larger tie actually increases ballast stress compared with a smaller tie. The large concrete tie generates higher deviatoric stress midway through the ballast layer than the medium synthetic tie. This is due to the high stress at the tie end that is generated when the tie stiffness is quite high relative to the roadbed stiffness. In this study, the large concrete tie was three times stiffer than the small concrete tie and five times stiffer than the wood tie. The "punch" effect of the stiff, large tie causes high stresses at the end of the tie on the relatively flexible roadbed with only 12 inches of ballast. Thus, crushing and flow of the ballast at the ends of the tie may be a problem. This loss of ballast support at the tie ends has been observed recently at the Facility for Accelerated Service Testing (FAST) track in Pueblo, Colorado. Loss of ballast at the tie ends may also be increased by vibration which is aggravated by tie centerbinding and rail corrugation.

The effect of too thin a ballast layer for a stiff tie should not be ignored. As the ballast depth is increased, ballast stiffness, and thus roadbed stiffness, increases. Therefore, the ratio between tie and roadbed stiffness decreases and the "punch" effect is reduced.

Figure 3-16 shows the maximum bulk stress at a location midway through the ballast for the same parameter variations discussed previously. The stress level reduces rapidly and converges to within 2 psi of a common value as ballast depth is increased from 12-in. to 36-in. Maximum bulk stress levels equivalent to, or less than, wood tie track can be achieved with several combinations of tie size, spacing, and ballast depth. The "punch" effect of using a large concrete tie with a thin ballast layer is evident. Increasing tie size without properly increasing ballast depth could minimize the advantages of a larger tie.

The maximum pressure on the ballast surface under ties is one of the criteria used in conventional track design procedures. See Table 4-2 for example. A maximum allowable pressure of 65 psi is typical. Figure 3-17 shows the maximum vertical ballast pressure predicted by MULTA. Increasing tie spacing increases

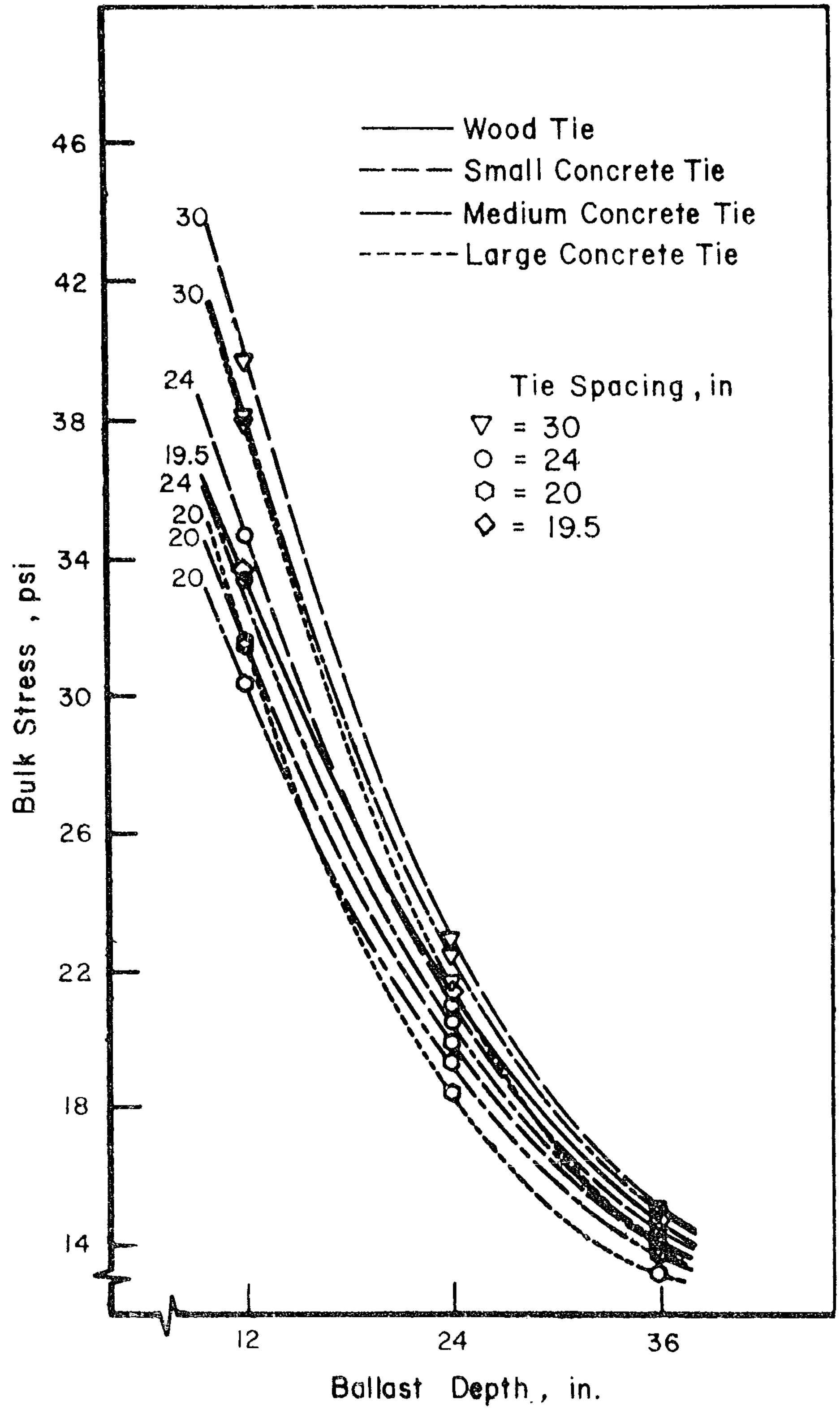


FIGURE 3-16. MAXIMUM BULK STRESS MIDWAY THROUGH BALLAST LAYER

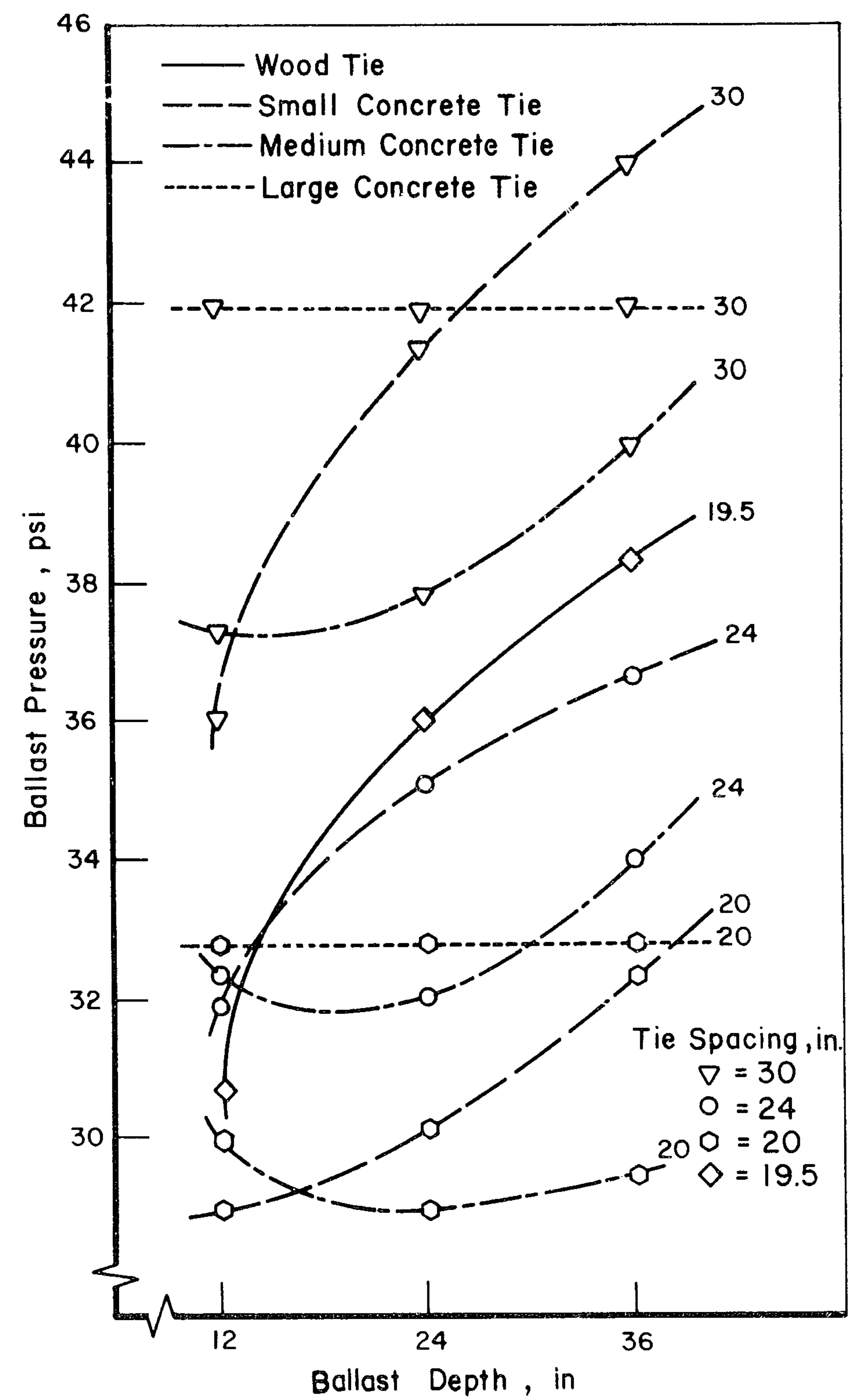


FIGURE 3-17. MAXIMUM VERTICAL BALLAST PRESSURE AT TIE/BALLAST INTERFACE

the ballast pressure in all cases, as expected. Increasing track stiffness by increasing ballast depth also causes some increase in ballast pressure due to the increase in rail seat load. However, the interaction between tie and roadbed stiffness, discussed previously, is also evident, and this obscures any clear trends. Increasing the roadbed stiffness with a relatively flexible tie, such as wood, or the small and medium size concrete ties, does increase the maximum ballast pressure. However, the ballast pressures from the stiff, large tie are independent of ballast depth and can provide some advantage over the smaller ties at the same spacing on deep ballast. This is due to the punch effect. That is, the vertical stress distribution for the large concrete tie approaches the case of a rigid tie on a flexible foundation for the ballast depths considered in this study. This being the case, the vertical stress value rises very sharply toward the end of the tie and it is difficult to discriminate values with such a sharp rise in stress locus. This is why a constant stress value appears for the ballast depths considered.

3.6.6 Subgrade Stresses

Figure 3-18 shows the maximum vertical subgrade stress at the ballast/subgrade interface. The maximum values of subgrade stress decrease rapidly with increased ballast depth. Vertical stress on the subgrade increases with a corresponding increase in tie spacing, but this effect can be offset by a small increase in ballast depth. The vertical stress converges to a common value for all tie sizes and spacings, for ballast depths greater than about 24 inches. There are many possible choices of tie size and spacing to equal or reduce the stress levels predicted for wood tie track.

Figure 3-19 shows the maximum subgrade deviator stress, which occurs at the ballast/subgrade interface. The maximum deviator stress is also very sensitive to increases in ballast depth and tie spacing. For ballast depths of about 12 inches, stress increases from increasing tie spacing can be offset by equivalent changes in ballast depth. For ballast depths greater than about 30 inches, the effects of changing tie size and spacing becomes negligible.

3.6.7 Ballast and Subgrade Stress Distribution

The previous data showed maximum stresses which occur at some point under a tie. Although the maximum stress is certainly a major factor in track

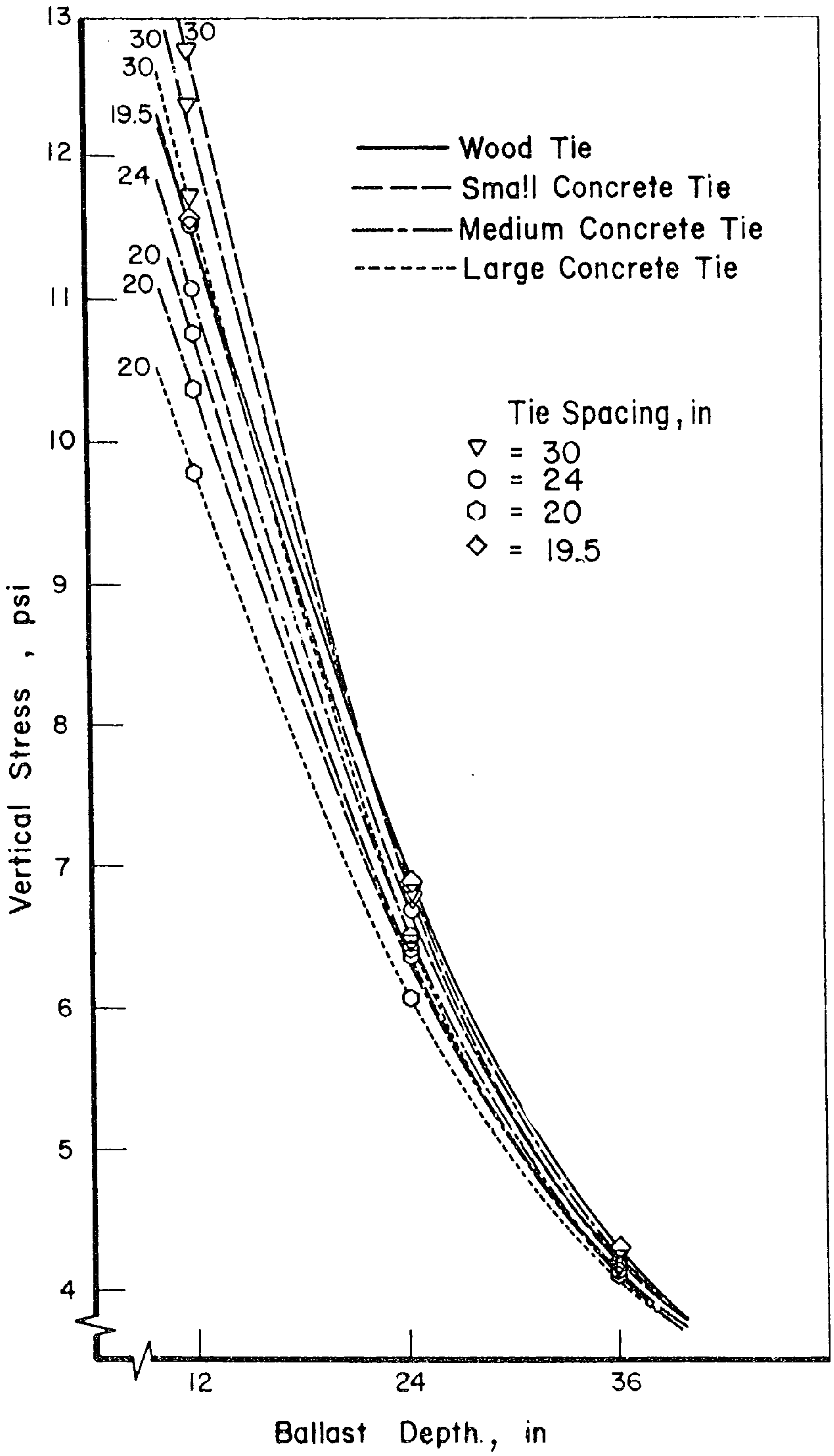


FIGURE 3-18. MAXIMUM VERTICAL SUBGRADE STRESS

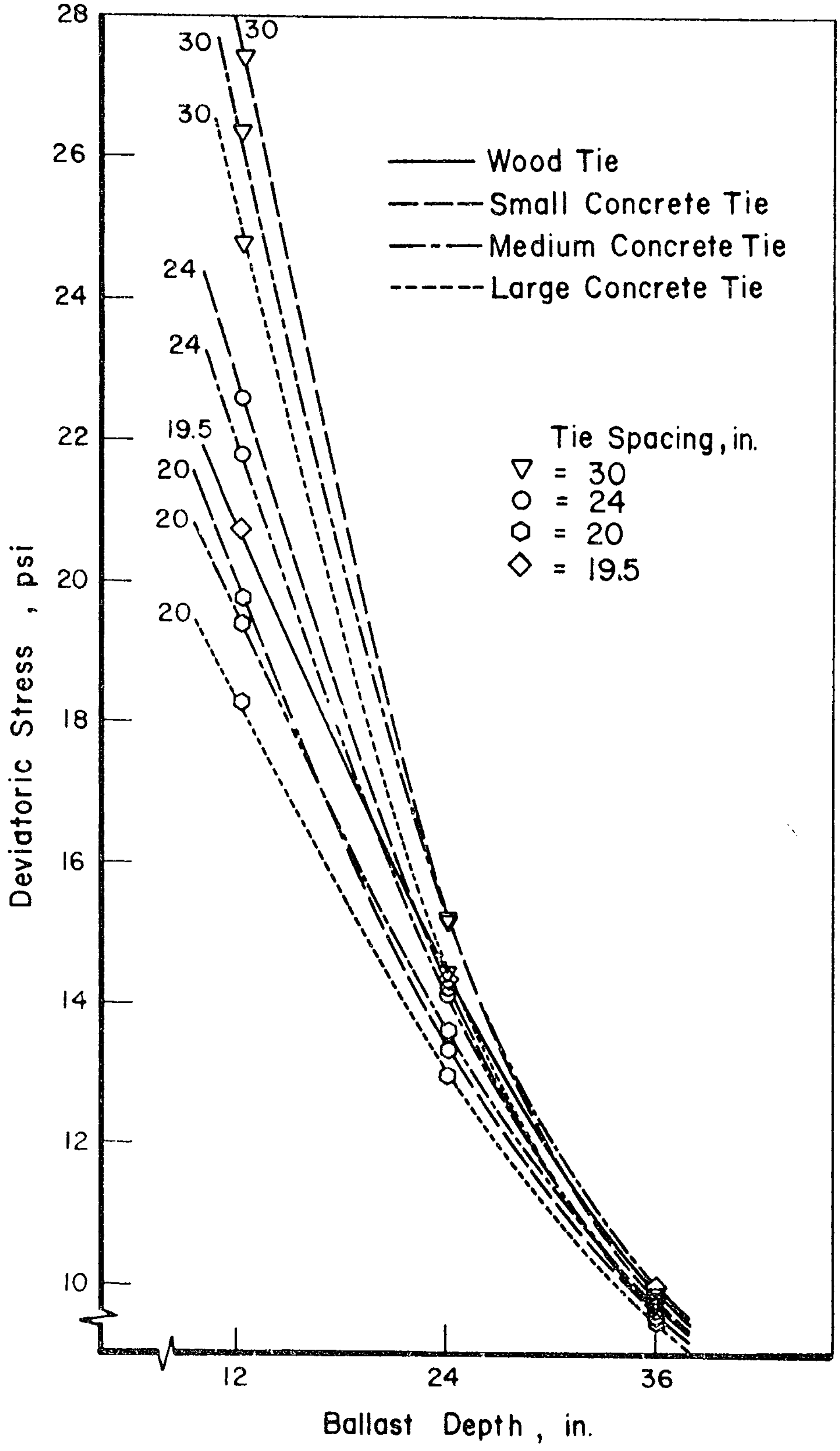


FIGURE 3-19. MAXIMUM SUBGRADE DEVIATORIC STRESS

performance, the variations in stress under a tie and along the track length also contribute to degradation. Non-uniform ballast stresses cause differential compaction and flow under the ties leading to an end-bound or center-bound support condition. Uniform ballast stresses would hopefully cause uniform settlement. This would minimize the effects of non-uniform support conditions and reduce the peak pressures to the average pressure, allowing effective use of the entire bearing area of the tie.

Non-uniform stresses on the subgrade cause depressions, or "rutting." These depressions will collect and retain moisture in climates with significant rainfall, and the resulting local reduction in subgrade strength will cause a rapid increase in the rate of settlement and pumping. This possibility was recognized by Salem and Hay [3-6], who recommended a ballast depth of about 18 inches to minimize subgrade pressure variations along the track with wood ties spaced at 21 inches. However, significant subgrade pressure variations remain under the tie even with ballast depths of 18 inches. This indicates that even greater ballast depths may be required to achieve a uniform pressure distribution and eliminate subgrade "rutting" under the rails. This has previously been ignored in track design.

Several pressure ratios have been calculated as a quantitative measure of roadbed pressure distribution uniformity. An indication of pressure variations may be had from the pressure ratio for the maximum to minimum variations under a tie, and for the maximum to minimum variations from under the tie to midway between ties at the rail seat region. A ratio close to 1.0 represents the ideal pressure distribution to minimize differential ballast degradation and rutting in the subgrade. Figures 3-20 and 3-21 show these two pressure ratios in the ballast for small concrete tie and wood tie track. The stiffer concrete tie produces a more uniform pressure distribution under the tie, Figure 3-20, for practically all tie spacing and ballast depth combinations, but the wood tie track shows more uniform pressures along the track, Figure 3-21. Increasing tie spacing causes a relatively large increase in pressure ratio along the track compared to under the tie. However, the pressure variations under the tie are higher and, as discussed previously, are therefore, the more critical design problem.

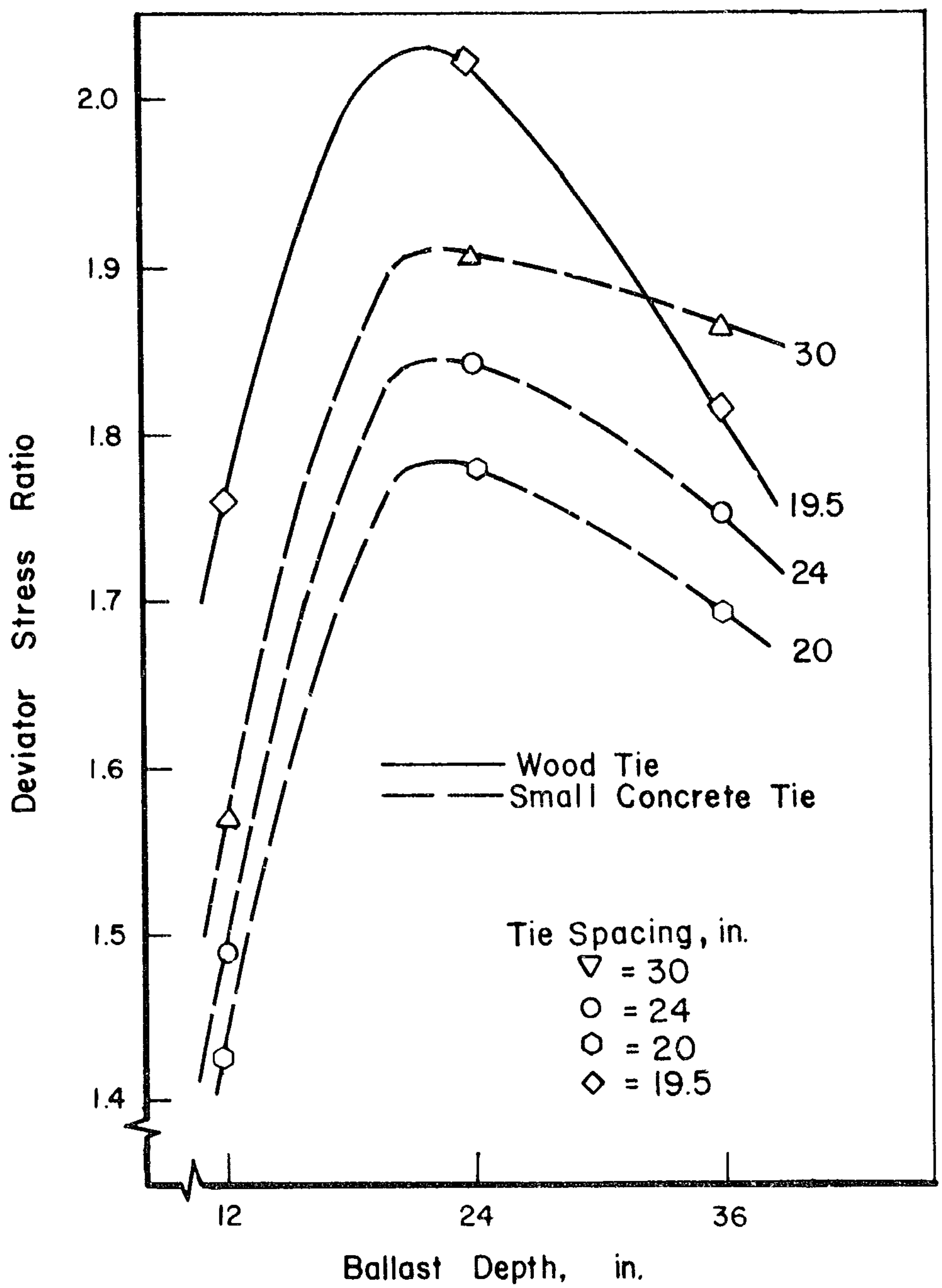


FIGURE 3-20. MAXIMUM/MINIMUM DEVIATOR STRESS RATIO UNDER THE TIE, ALONG TIE LENGTH MIDWAY THROUGH BALLAST

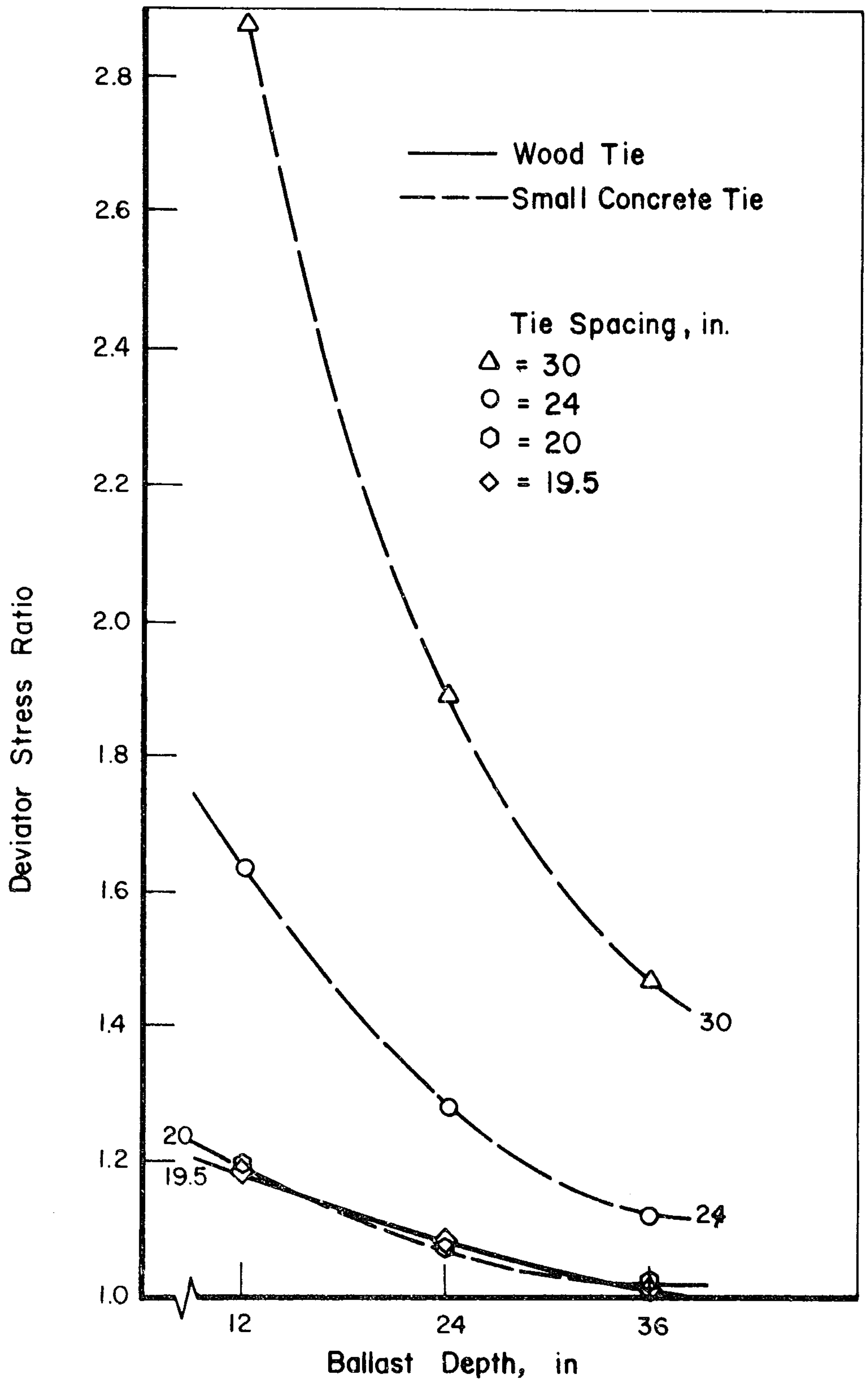


FIGURE 3-21. MAXIMUM/MINIMUM DEVIATOR STRESS RATIO ALONG TRACK MIDWAY THROUGH BALLAST

Figures 3-22 and 3-23 show the same pressure ratios at the subgrade as a measure of potential rutting. Increasing ballast depth effectively attenuates subgrade pressure variations both under the tie and along the track. The effects of increasing tie spacing are relatively minor for ballast depths greater than about 24 inches. The subgrade pressure variations under the tie are more important than those along the track for ballast depths greater than about 12 inches.

3.6.8 Effect of Rail Fastener Stiffness

The results presented in the previous sections were based on a rail fastener having a nominal vertical stiffness (spring rate) of 40×10^5 lb/in., typical of many fasteners currently being used with concrete ties in the U.S. This stiffness represents the total load-deflection characteristics for a rail fastener assembly consisting of rail restraining devices and a tie pad. Rail fasteners are simulated in the MULTA program by linear vertical springs between the rail base and each tie.

Figure 3-24 shows that reducing the rail fastener stiffness increases rail displacements significantly when the fastener stiffness is less than about 500,000 lb/in. This reduction in stiffness also distributes the wheel load over more ties so that the maximum rail seat loads, and therefore tie deflection are reduced. The effect of varying the rail fastener stiffness depends on the stiffness of the fastener relative to the effective roadbed stiffness at each tie. When the fastener is rigid relative to the roadbed, the track response is governed by the roadbed stiffness and the deflection of the rail fastener is very small, as shown in the right side of Figure 3-24. When the fastener is very flexible relative to the roadbed, the track response is governed by the fastener stiffness, as shown in the left side of Figure 3-24.

Figures 3-25 and 3-26 show that a flexible rail fastener does reduce the maximum rail seat load, and therefore the tie bending moments and the tie/ballast pressure. The maximum subgrade pressures would be reduced accordingly. Rail bending moments are increased, but this is not usually critical unless a relatively small rail is used.

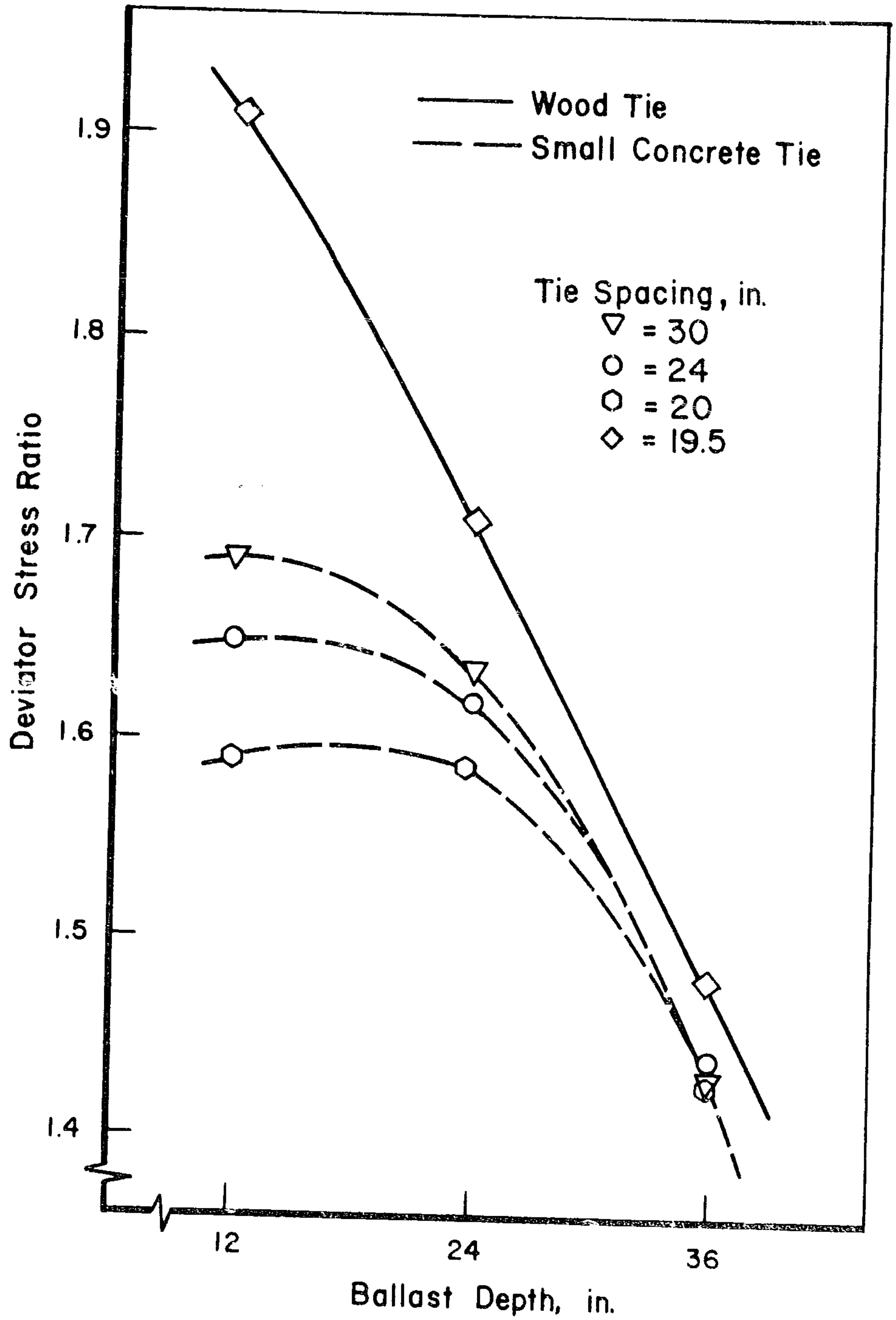


FIGURE 3-22. MAXIMUM/MINIMUM DEVIATOR STRESS RATIO UNDER TIE AT BALLAST/SUBGRADE INTERFACE

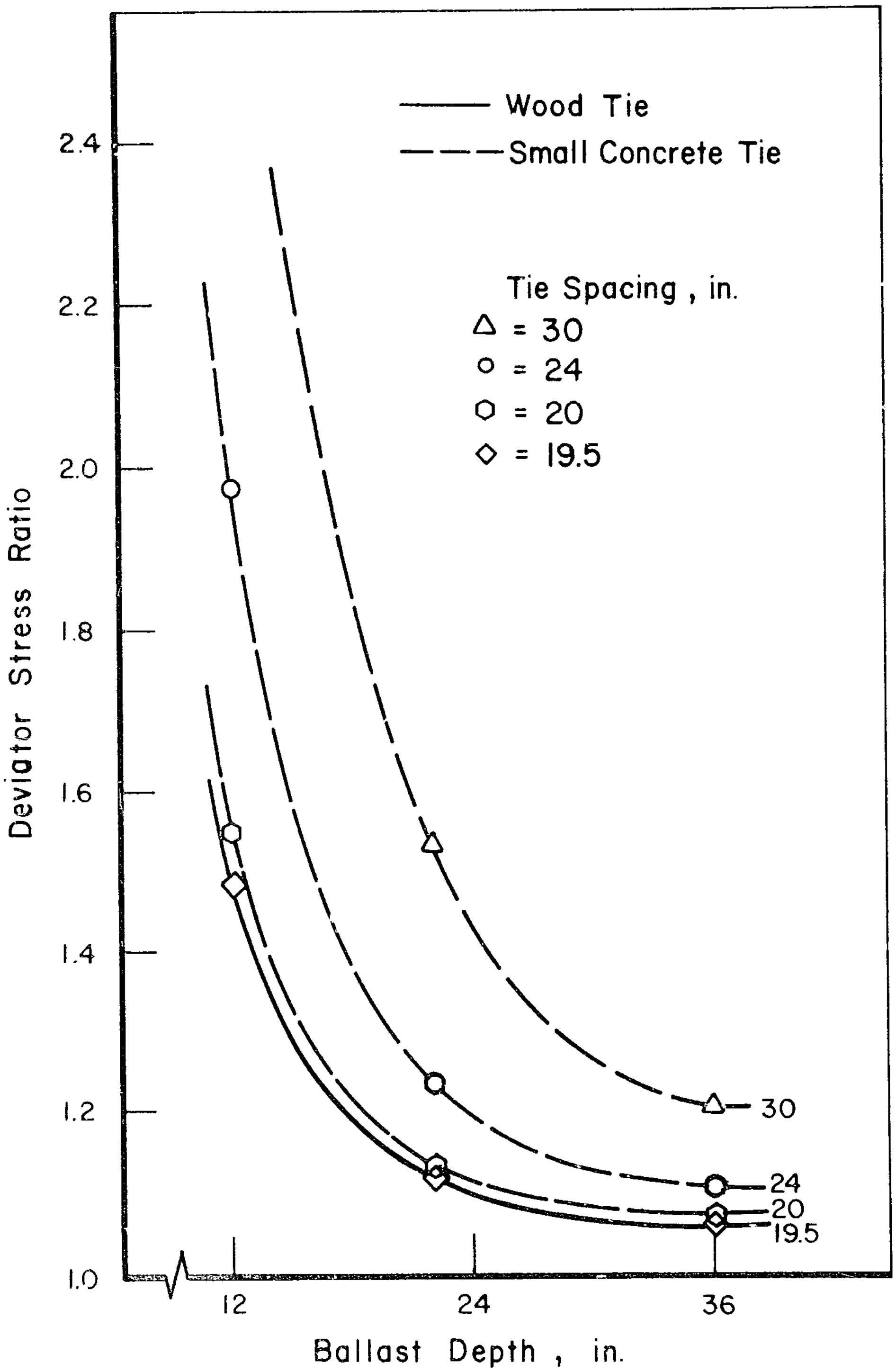


FIGURE 3-23. MAXIMUM/MINIMUM DEVIATOR STRESS RATIO ALONG TRACK AT BALLAST/SUBGRADE INTERFACE

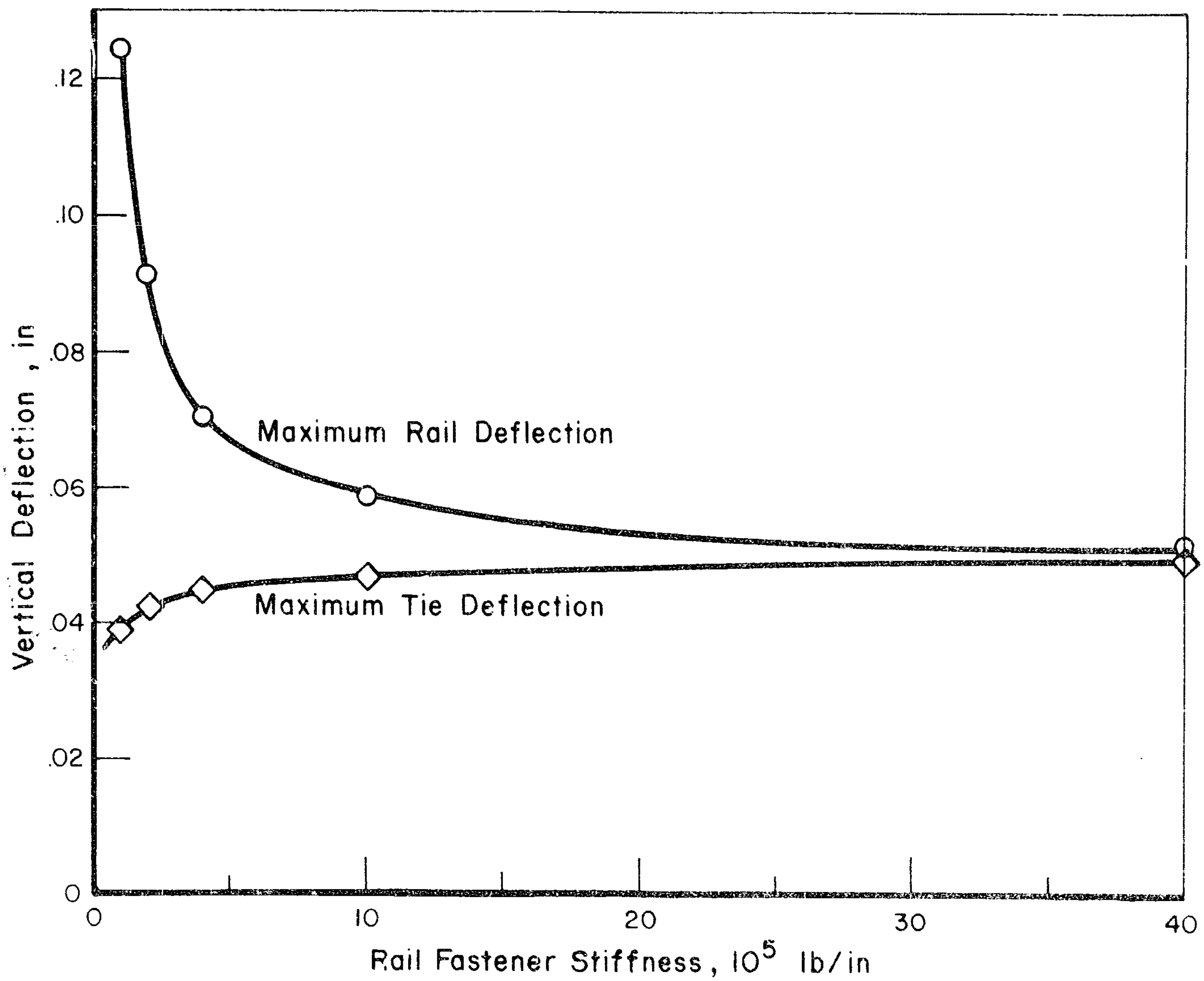


FIGURE 3-24. EFFECT OF RAIL FASTENER STIFFNESS ON MAXIMUM RAIL AND TIE DEFLECTIONS

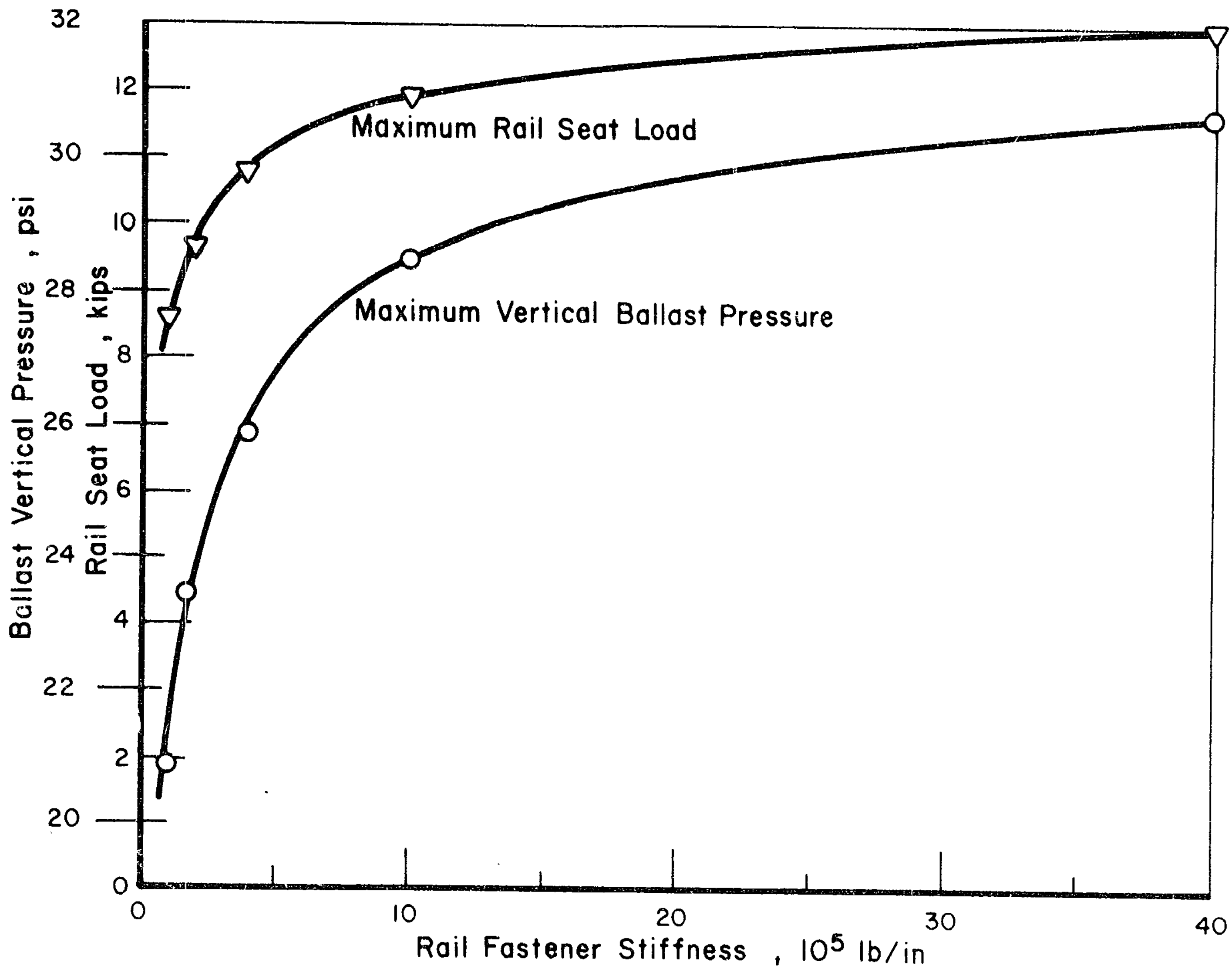


FIGURE 3-25. EFFECT OF RAIL FASTENER STIFFNESS ON MAXIMUM VERTICAL RAIL SEAT LOAD AND BALLAST PRESSURE

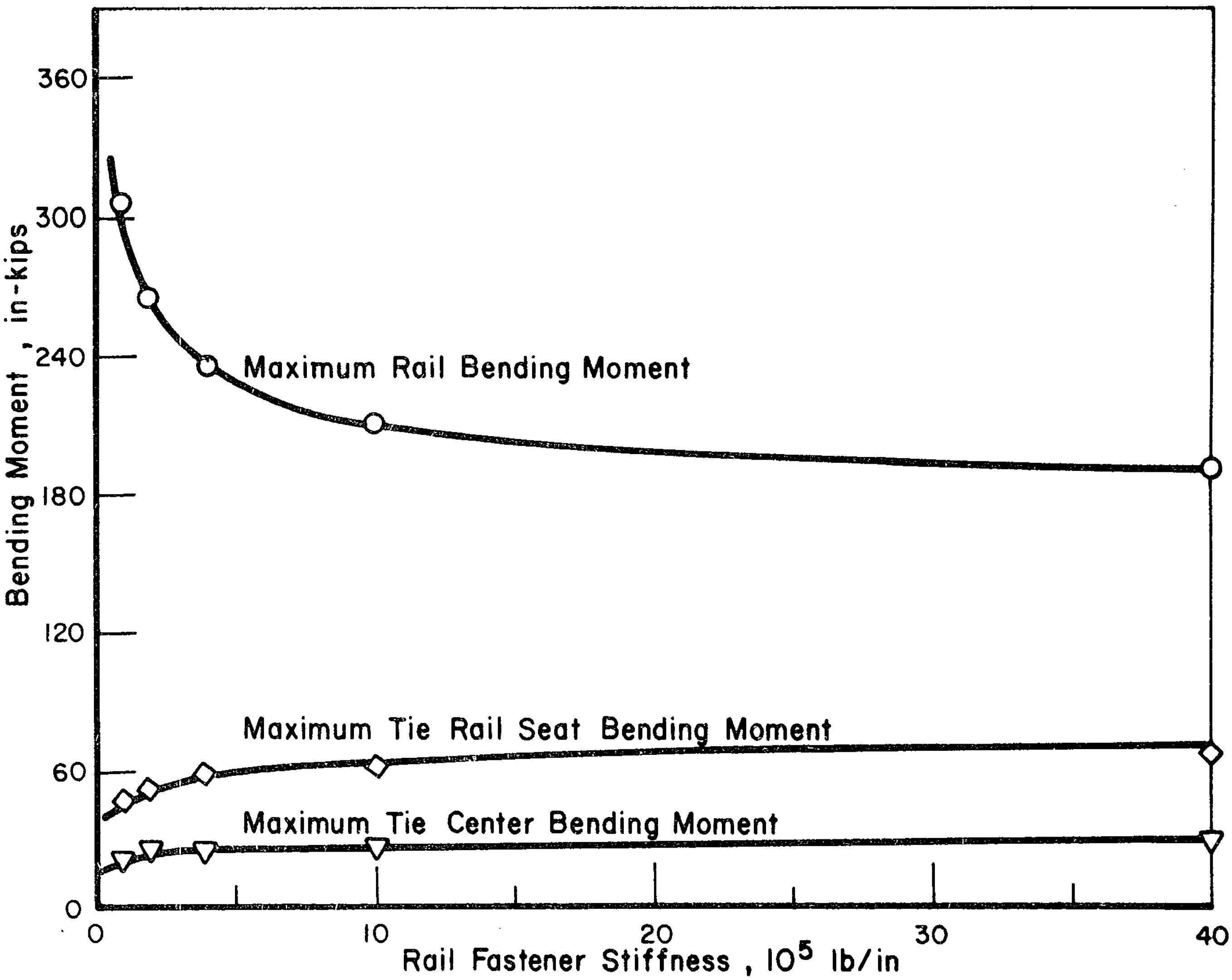


FIGURE 3-26. EFFECT OF RAIL FASTENER STIFFNESS ON RAIL AND TIE BENDING MOMENT

It is important to realize that practically all of the rail fasteners currently being used for concrete ties in the U.S. are relatively rigid compared to the roadbed. Table 3-3 shows some typical stiffness for three general classes of fastener. Very flexible fasteners are used for direct fixation transit track, such as Toronto, where it is important to attenuate ground vibrations in subways. These fasteners restrain the rail by thick pads of

TABLE 3-3. TYPICAL RAIL FASTENER VERTICAL STIFFNESS DATA

Fastener Type	Toe Load, kips per clip	Clip Stiffness, 10^5 lb/in. per clip	Total Fastener Stiffness, 10^5 lb/in.
Very flexible	--	--	1 - 2
Flexible	1.6 - 2.7	0.02 - 0.07	5 - 16
Rigid	--	0.2 - 5.0	20 - 70

rubber and are sufficiently flexible to reduce tie loads. But, they are not designed for the higher axle loads of U.S. railroads.

The flexible fastener category includes several different configurations with metal retaining clips having considerable flexibility. However, these are generally installed in the U.S. with a relatively thin (1/8 - 3/16) rubber or plastic rail pad that is very stiff relative to the clip. This produces a fastener with a vertical stiffness of 5×10^6 lb/in. to 16×10^5 lb/in., which provides very little reduction of track loads. The stiffness of the rail pad determines the total stiffness of the fastener assembly for these designs.

Rigid fasteners include several configurations of stiff metal clips with thin rail pads of very hard material. The rail pad must be quite stiff to avoid fatigue failures of the rigid clip and attachment hardware. The stiffness of these fasteners is typically in the 20×10^5 lb/in. to 70×10^5 lb/in. range, and this cannot be expected to produce any substantial reduction of static or dynamic rail loads.

Thus, rail fasteners must have a vertical stiffness less than about 500,000 lb/in. in order to provide any significant benefit by distributing wheel loads over more ties. These conclusions are based on a static analysis where the load is assumed to be constant. A flexible fastener would also reduce dynamic loads resulting from track irregularities, such as joints or rail welds, and wheel flats. The increased stiffness of concrete tie track is undesirable because it produces higher dynamic forces which adversely affect maintenance of both track and vehicles. Previous studies by Battelle [3-7, 3-8] and others have shown that it is desirable to introduce resilience into the rail fastener to compensate for this increased stiffness. Development efforts in Europe and Russia of fasteners having multiple thick elastomeric pads indicates that increased flexibility is a major design objective. This trend appears to have been ignored in the U.S., where recent fastener modifications have included reducing the thickness and increasing the durometer of pads to improve fatigue life of the rail clips--all steps which increase fastener stiffness.

Once fastener resilience is given a high design priority, achieving a successful design is no small challenge. The major problem is maintaining adequate lateral restraint against gage spread and rail rollover while reducing vertical stiffness. Another problem is that stiffness characteristics of most elastomers vary considerably with temperature and probably load, making it difficult to maintain uniform performance throughout the year. However, the reduction of impact loads and the improvement in load distribution which can be obtained with more flexible fasteners should be adequate to encourage additional development efforts by industry.

3.7 DESIGN FOR EQUIVALENT TRACK PERFORMANCE

It is difficult to compare different track structure designs in a meaningful way because we lack criteria to relate track response parameters in the form of ballast and subgrade stresses to quantitative predictions of track degradation rate. However, it is possible to select track structures expected to give equal performance with regard to surface maintenance by comparing selected track response parameters from the parametric study.

For example, Figure 3-27 shows those concrete tie track designs which have the same maximum subgrade deviatoric stress as wood tie track with 19.5 in. tie spacing and 12 inches of ballast. All calculations are based on 136 lb/yd rail and the same subgrade and material properties used for the parametric study. These results show that relatively small increases in ballast depth will compensate for substantial increases in tie spacing.

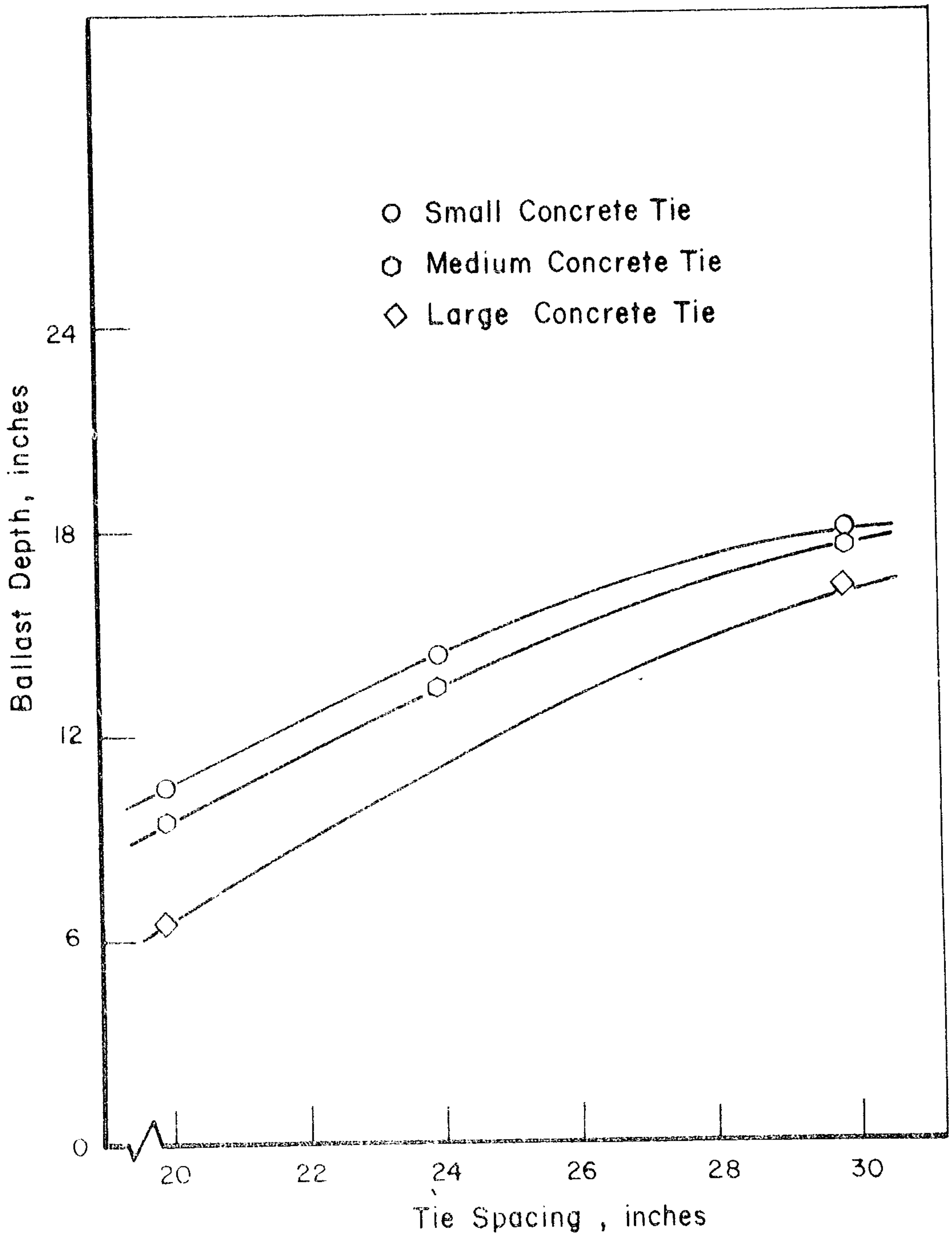


FIGURE 3-27. TIE SPACING AND BALLAST DEPTH REQUIREMENTS TO GIVE SAME MAXIMUM SUBGRADE DEVIATOR STRESS AS WOOD TIE TRACK

Table 3-4 summarizes the equivalent track design parameters for several tie spacings based on equal subgrade and ballast stresses. Equivalent designs are shown based on maximum vertical stresses as used in conventional design procedures and maximum deviator stress which is recommended as a more suitable indication of long-term performance.

This comparison shows that the greatest ballast depths for a selected tie spacing will be required to equalize the ballast deviator stress. Both ballast and subgrade deviator stress criteria require greater ballast depths and, therefore, a more conservative design than the vertical stress criteria used for conventional track design. These summary data also show that the larger concrete tie has an advantage of requiring less ballast depth or wider spacing for comparable performance. This results primarily from the increase in bending stiffness, because the tie width is almost the same for the small, medium, and large concrete ties. A substantial increase in width would give an added performance advantage to the large tie.

As an example of the use of Table 3-4, suppose one is interested in choosing a concrete tie track system that is expected to have performance equal to the wood tie track simulated in this parametric study. If one chooses a small concrete tie with 24-in. spacing, a ballast having a 16 in. depth is required to insure that the concrete tie track performance will equal or exceed that of the wood tie track. This choice of ballast depth shows that subgrade vertical stress, subgrade deviation stress, and ballast vertical stress levels will be below the corresponding stress levels for the wood tie track structure. Alternatively, if one picks a large concrete tie track with 24 in. spacing, then only 12 in. of ballast are required to insure equal concrete and wood tie track performance. There are several choices of tie size and spacing from Table 3-4 that will give equal concrete and wood tie track performance. The final choice of tie size and spacing and corresponding ballast depth requirements should be made based on the economic aspects of track installation and maintenance. A life cycle cost analysis for wood and concrete tie track is planned for a later phase of this project.

As discussed previously, increasing ballast depth also reduces the subgrade pressure variations as measured by the pressure ratios under the tie and along the track. An exercise similar to that shown in Table 3-4 could be done using equal pressure ratios as the criteria. The difficulty is that an

TABLE 3-4. COMPARISON OF CONCRETE TIE TRACK DESIGNS EXPECTED TO HAVE PERFORMANCE EQUAL TO STANDARD WOOD TIE TRACK (1)

Tie Type	Tie Spacing, in.	Ballast Depth, in.			
		Equal Subgrade Vertical Stress (2)	Equal Subgrade Deviator Stress (3)	Equal Ballast Surface Vertical Stress (4)	Equal Ballast Deviator Stress (5)
I. Small Concrete Tie	21.5	11	12	17	10.5
	24	12	14.5	10.5	16
	27	13	16.5	9	21
	30	14	18	9	24.5
II. Medium Concrete Tie	21.5	9.5	11	--	7
	24	10.5	13.5	--	13.5
	27	11.5	16	--	19
	30	12.5	17.5	--	23.5
III. Large Concrete Tie	21.5	7.5	8.5	--	8.5
	24	9	11	--	12
	27	10.5	14	--	17
	30	12.5	16	--	22

(1) 7-in. x 9-in. x 102-in. wood ties at 19.5 - in. spacing for wood tie reference track.
(2) Maximum subgrade vertical stress = 11.6 psi.
(3) Maximum subgrade deviator stress = 20.7 psi.
(4) Maximum ballast vertical stress = 30.7 psi.
(5) Maximum ballast deviator stress midway in ballast layer = 21.1 psi.

overall performance index which combines these different parameters with appropriate weighting factors for track degradation is needed to further quantify track design.

A previous report [4-1] under this contract included a review of the principal modes of track component failure and long-term track degradation. This was used to identify criteria for selecting track analysis models to predict the governing response parameters such as tie bending moments, fastener loads, and ballast and subgrade pressures. The general formats for several performance measures were identified. An additional development of quantitative performance indices for track degradation is discussed in this report.

The previous review of track failure modes included:

A. Failure Due to Non-Retention of Track Geometry
   a. Track surface (profile and cross level) deterioration
   b. Track alinement deterioration
   c. Wide gage
   d. Rail rollover
   e. Track buckling and track shift.

B. Component Failure
   a. Rail failure
   b. Tie failure due to bending and torsion
   c. Rail fastener and pad failure.

These degradation modes and governing performance measures were discussed in [4-1], and will not be repeated here. However, Table 4-1 has been reproduced to summarize the performance indices and critical track response parameters for the major failure modes. These criteria were used to select specific track models for vertical and lateral analyses. A track model MULTA for MUlti-Layer Track Analysis was recommended for vertical response analysis, and extensive data from this model have been discussed in Section 3 of this report. The need for development of a 3-D lateral track model was identified, but this task was not undertaken as a part of this contract. Subsequent analysis efforts have been concentrated on track response to vertical loads because this represents the highest priority for evaluating concrete or synthetic tie track where rail rollover, wide gage and lateral buckling problems are minimized by the rigid rail fasteners. Because of this emphasis on vertical track response, subsequent work on track performance measures was concentrated on the relationship between track response and the deterioration of track surface.

TABLE 4-1. SUMMARY OF TRACK DEGRADATION MODE ANALYSIS REQUIREMENTS

Degradation Mode	Performance Index	Critical Parameters	Analysis Model Requirements	Load Requirements	Frequency Range, Hz
1. Tie failure from bending and torsion	Number of expected tie failures per mile per MGT based on probability of fracture or exceeding fatigue life	Tie bending and torsion moment probability density for specified traffic. Tie strength probability density for static load failure. Fatigue statistics for tie bending and torsion moments and statistical description of tie fatigue strength in torsion and bending	Single vertical tie finite element model with rail seat loads and moments and variable stiffness ballast support to predict tie bending moments. Estimate of maximum torsional moment based on predicted statistical tie plate loads	Probability distributions of peak vertical rail seat loads and rollover moment for the specified traffic, speeds, and track condition	0-50
2. Rail fastener failure a) Pull-out of tie inserts b) Failure of rail clips	Number of expected fastener component failures per mile per MGT based on probability of fracture or exceeding fatigue life	Probability density of maximum rail seat loads L/V & M_r/V at rail base. Probability density of fastener component strength for static load failure. Statistical description of fastener component fatigue strength	3-D finite element track model which includes non-symmetrical vertical and lateral W/R loads, fastener stiffness, rail torsion and non-linear stiffnesses for fastener and ballast	Probability density of peak vertical, lateral and L/V W/R loads and peak rail seat loads and moments	0-2000
3. Track surface deterioration (vertical profile and cross level) a) Ballast failure and flow b) Subgrade failure and settlement.	Rate of rail profile and cross level deterioration versus wavelength	Probability density of maximum and average tie ballast pressure, maximum subgrade deviator stress (σ_1-σ_3), cumulative settlement data for ballast and subgrade materials	Vertical track model using Burmister's multi-layer roadbed model and load distribution program to predict ballast and subgrade pressures and tie deflections	Probability density of peak vertical W/R and rail seat loads for specified traffic	0-50
4. Track alinement deterioration	Number of occurrences per mile where critical load for track lateral shift is exceeded	Probability density of maximum lateral force ratio H/P for individual axles. Porbability density of track critical lateral force ratio H_c/P	Vertical track model using Burmister's multi-layer roadbed model and load distribution program to predict vertical tie loads. 2-D finite element lateral track model with thermal loads, rail fastener torsional resistance and nonlinear ballast resistance which is dependent on vertical tie loads	Joint probability density of peak vertical and H/P axle load ratio for specified traffic	0-10
5. Rail rollover	Probability of exceeding critical rail loading condition (L/V)	Probability density of maximum lateral force ratio (L/V) for one rail. probability density of rail critical L/V for rollover	3-D finite element track model which includes non-symmetrical vertical and lateral W/R loads, fastener stiffness, rail torsion and non-linear stiffnesses for fastener and ballast	Probability density of lateral/vertical load ratio for all wheels on one side of one truck	0-10
6. Wide gage	Probability of exceeding critical ratio for tie plate loading on wood ties. Probability of exceeding tie compressive strength on field side of tie plate. Rate of gage change per MGT.	Probability density of maximum L/V load ratio and maximum M_r/V moment ratio for tie plates. Probability density of allowable L/V and M_r/V for tie plate slip and tie crushing on wood ties	3-D finite element track model which includes non-symmetrical vertical and lateral W/R loads, fastener stiffness, rail torsion and non-linear stiffnesses for fastener and ballast	Probability density of L/V load ratio for individual wheels	0-50

H = Lateral axle force L = Lateral wheel force M_r = Rail rollover moment
P = Vertical axle force V = Vertical wheel force

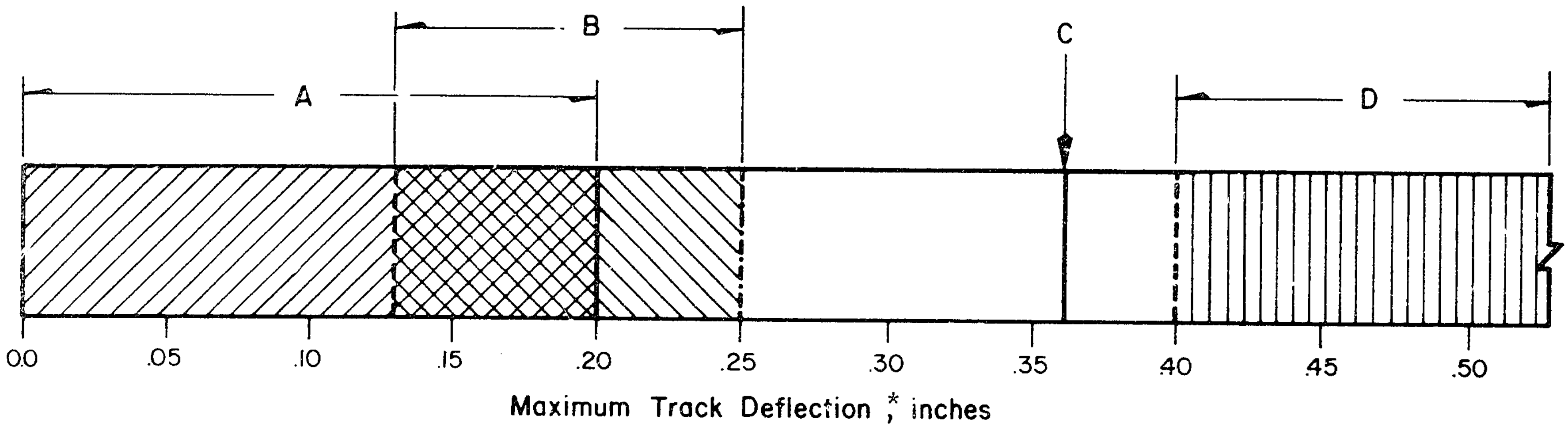
4.1 TRACK SURFACE DETERIORATION

The deterioration of track surface is determined by the differential vertical settlement of each rail (rail profile) and the differential settlement between rails at the same location (cross level). Surface maintenance is particularly prevalent on bolted-joint track. However, only continuous welded-rail (CWR) track is being considered in this effort, because CWR will usually be used for track construction with synthetic ties. Track settlement in the vicinity of structures, such as bridges or highway grade crossings, is also a perpetual problem. Some settlement relative to a fixed structure is inevitable, and this causes an abrupt change in track surface. However, the general deterioration of the surface of CWR track that is constructed on what would normally be considered a uniform roadbed is of principal concern for this project.

4.2 REVIEW OF PERFORMANCE CRITERIA FOR TRACK SURFACE DETERIORATION

A report [4-2] reviewing current track design procedures indicated that although track geometry is a key parameter in track performance, there are no design criteria directly related to the degradation of track surface from differential settlement along the track route. A general practice in track design is to prepare the roadbed to a minimum acceptable soil-bearing capacity. In some cases, cost/benefit analyses have been used to evaluate improving subgrade capacity versus increased ballast depth or decreased tie spacing. Then, a uniform track construction in terms of ballast depth, tie spacing, and rail size are selected using past experience and analytical predictions of track deflection and average ballast and subgrade pressures. This results in a track which can have considerable variation in stiffness and strength from one location to another; hence, differential settlement can be expected.

The AREA recommendation [4-3] of a maximum track deflection of 0.25 inches, based on the beam-on-elastic-foundation analysis procedure has been used for recent design evaluations [4-4] of new track construction for the Northeast Corridor (NEC). Figure 4-1 from Reference [4-5] shows similar track deflection criteria based on Talbot's studies for the AREA Special



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

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

FIGURE 4-1. TRACK DEFLECTION CRITERIA FOR DURABILITY OF WOOD TIE TRACK [4-5]

Committee on Stresses in Railroad Track. Although deflection criteria are undoubtedly based on experience with typical wood tie track, a performance index based on deflection appears to have little justification for track design.

Typical recommendations for maximum pressure on the ballast vary considerably, as shown in Table 4-2. In accordance with current track design analysis, these pressures should be interpreted as the average pressure over the effective bearing area of the tie. The maximum (peak) ballast pressure under a tie would be considerably higher due to tie bending and variations in ballast compaction and reaction loads across the tie bottom.

Table 4-3 shows typical recommendations for safe average bearing pressures on the subgrade. Clarke [4-6] recommends a maximum pressure of 12 psi for uncompacted roadbed and 20 psi or more, depending on the soil, for compacted roadbeds. One design practice is to specify a ballast depth which would limit subgrade pressure to 60 percent of the "safe average bearing pressure" to account for variations in track uniformity. Alternatively, the rail seat load used for predicting pressures, for comparison with a soil allowable pressure, can be increased to account for track variations, as indicated by the AREA recommendation in Table 4-3 to double calculated rail seat loads.

Several different approaches for establishing track surface deterioration criteria are reviewed in the following sections.

4.2.1 The BR Approach to Ballast Depth Criteria

Some recent work [4-7] by Heath, et al., from British Railways (BR) represents some improvement on what is still a deterministic approach to track design. Triaxial tests under repeated loading of clay soil samples were used to obtain cumulative strain (settlement) data versus number of loading cycles. Those results for clay indicated an effective threshold stress difference (endurance limit) such that when the endurance limit stress is exceeded, subgrade settlement continues at a high rate and is roughly proportional to the logarithm of the number of loading cycles. However, when the stress difference from the applied load is less than the threshold stress, the cumulative settlement reaches equilibrium and very little additional settlement occurs. The assumptions used for a design method based on these results are as follows:

TABLE 4-2. TYPICAL VALUES FOR MAXIMUM ALLOWABLE BALLAST PRESSURE

Source Description	Pressure, psi
Allowable pressure to avoid crushing of good stone ballast, Clarke [4-6]	35
AREA recommendation based on typical pressures under wood ties at 20-inch spacing [4-3]	65*
Recommended for concrete ties on high-quality, abrasion-resistant ballast, AREA [4-8]	85

*The AREA recommends that the rail seat loads used to determine maximum permissible tie spacing based on ballast pressure be doubled to allow for normal variances in tie support conditions.

TABLE 4-3. TYPICAL VALUES FOR SAFE AVERAGE BEARING PRESSURES FOR TRACK SUBGRADES

Source Description	Pressure, psi
Clarke [4-6]	
Alluvial soil	<10
Made ground not compacted	11-15
Soft clay, wet or loose sand	16-20
Dry clay, firm sand, sandy clay	21-30
Dry gravel soils	31-40
Compacted soils	>41
AREA recommendation [4-3]	20*

*The AREA recommends that the rail seat loads used to determine subgrade pressures be doubled to allow for normal variances in tie support conditions.

(1) The threshold stress parameters for the subgrade soil may be obtained using a triaxial repeated load test.

(2) Simple elastic theory using a single elastic layer model gives adequate predictions for subgrade stresses. Measured data showed a linear relation between tie loading and subgrade stress independent of vehicle speed. The flexural stiffness of the tie was of secondary importance and subgrade stresses under wood and concrete ties were very similar. The wide scatter in results from variations in tie support and ballast compaction condition does not warrant a more rigorous approach.

(3) The significant stresses are those produced only by the static effect of the heaviest frequently occurring axle loads, thereby neglecting any cumulative effects from lighter axles.

(4) The water table is at the top of the subgrade, so a saturated specimen is used for the subgrade triaxial tests. This condition is responsible for the most severe track degradation, but it may only occur a few times each year.

The design method is based on using a ballast depth so that the calculated maximum principal stress difference (deviatoric stress) in the subgrade for the heaviest axle load is equal to the average threshold principal stress difference from laboratory soil tests. Track measurements have shown that reduced settlement rates are achieved consistently with the required depth of ballast. This design procedure typically gives recommended ballast depths in excess of 30 inches. Settlement rates for track with less ballast depth are significantly higher. Typical settlement rates of 3/4 to 1-1/2 inches per million axles were recorded for wood tie track having ballast depths from 4 to 8 inches less than the design goal which is based on the subgrade threshold stress. Typical rates of 1/4 to 3/4 inches per million axles were observed for ballast depths from 4 to 8 inches thicker than the design goal, with a rapidly diminishing return for greater depths. Presumably much of this settlement occurs in the ballast if the subgrade settlement has truly been reduced significantly.

British Railways recognizes that a complete design procedure using the elastic approach requires knowing:

(a) The loading spectra in terms of amplitude and frequency of occurrence

(b) the stress-strain distribution and its variability through the complete track structure

(c) the subgrade material properties and variability for relevant loading and environmental conditions.

British Railways indicates the ultimate need for a probabilistic approach to track design based on a finite rate of degradation that is cost-effective. Also, the design approach used neglects the degradation and settlement of material used for ballast and subballast, even though it recognizes that only the top layer of what may be 3 to 4 feet above the subgrade needs to be high quality ballast to resist high ballast pressures and abrasion at the tie/ballast interface. The general design philosophy is that although differential vertical settlement (the factor which governs surface maintenance) is not predicted directly, it will be proportional to total vertical settlement. Hence, a reduction in verticle settlement rate will reduce differential settlement by a corresponding amount.

4.2.2 A Track Settlement Model

A more direct approach to predicting track differential settlement is described in TRW studies [4-9, 4-10]. A track structure settlement model and computer program were developed based on a nonlinear finite element model for the roadbed. The ties, ballast, and soil were represented by a series combination of a linear spring, a hysteretic spring, and a damped hysteretic spring, respectively, as shown in Figure 4-2. The train loading included a static wheel load plus as many as three sinusoidal components to represent dynamic wheel loads, but all feedback coupling between track response and vehicle dynamic wheel loads was neglected to simplify the analysis. The computed results included the residual settlement of the ballast and subgrade for a single train passage. The cumulative effect of several trains was then determined as a function of the settlement rate for a single train.

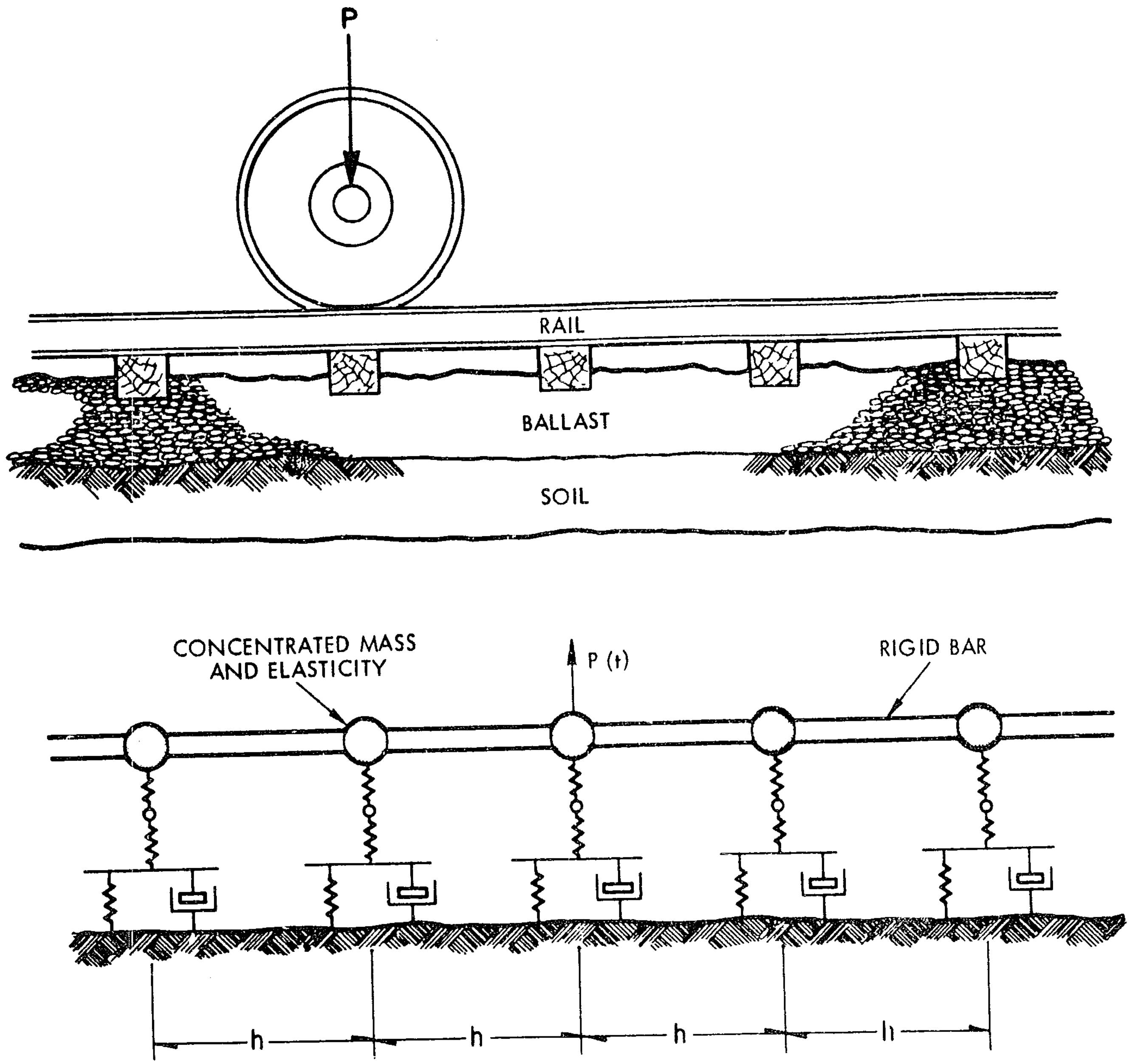


FIGURE 4-2. ROADBED MODEL [4-10]

and residual deformation of the ballast and subgrade under repetitive loads. The ballast model was based on data for coarse gravel, because no data were available for actual ballast materials. The load-settlement models shown in Figure 4-3 were admittedly quite simplified, and they were not expected to give accurate predictions of track settlement. However, they were judged to be adequate for parametric studies of the relative effects of different track stiffnesses, vehicle suspensions, weights, speeds, etc.

In Figure 4-3 the soil characteristics were simplified by assuming one constant modulus for the loading portion of the cycle and another modulus for the unloading portion. The loading modulus is taken to have the same value for all loading cycles. The unloading modulus, however, increases for each succeeding cycle in such a way that the residual deformation approaches zero as the number of cycles increases.

The right side of Figure 4-3 shows the deformations which occur as a soil element is loaded through repeated cycles. The left side of the figure shows how the permanent deformations build up as the load cycles occur. The behavior of the soil is described by the two constants a_2 and a_3, where a_2 is the deformation per load P for each cycle and a_3 is the change in permanent deformation per 10 cycles.

This settlement model was only used to predict the uniform settlement for uniform track properties. Some simplified assumptions were made to relate differential settlement (track roughness) to the calculated uniform settlement because of the lack of available data relating the probable variation of soil and ballast properties along typical track. Figure 4-4 shows how this approach was used to estimate a roughness versus wavelength relationship. A similar procedure was used to estimate horizontal displacements.

4.2.3 Ballast and Subgrade Material Elastic Properties

Any track analysis methodology expected to give useful predictions of track deterioration must include a realistic representation of the roadbed materials. The material parameters which govern the behavior of ballast and subgrade under cyclic loading are quite complex. The elastic properties of

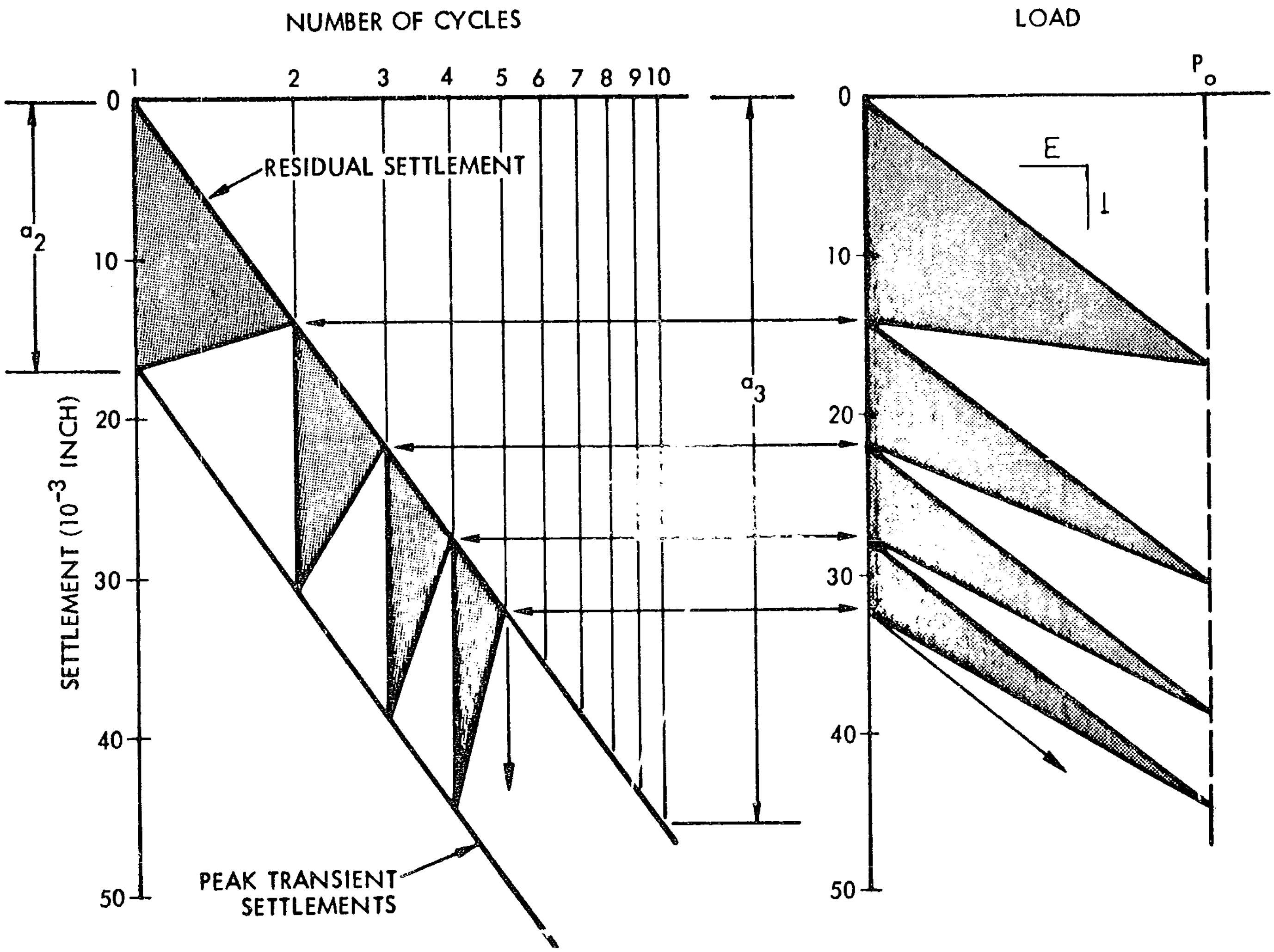


FIGURE 4-3. SOIL LOAD SETTLEMENT CHARACTERISTICS [4-10]

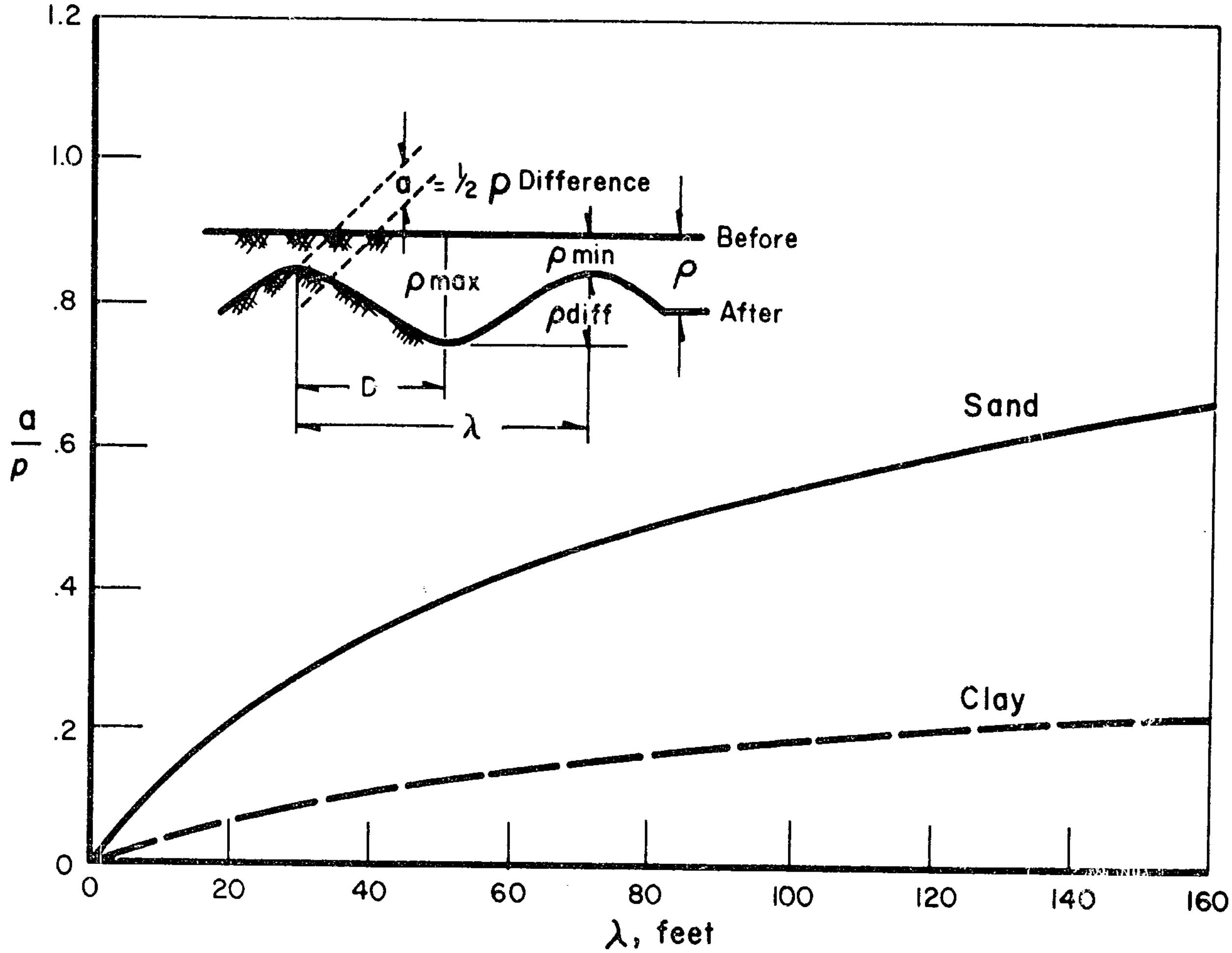


FIGURE 4-4. UNIT DIFFERENTIAL SETTLEMENT VERSUS WAVELENGTH [4-10]

granular materials are stress dependent and are typically characterized by a resilient modulus E_R (repeated deviator stress/elastic strain). The resilient modulus for granular materials increases as a function of the sum of the principal stresses, $\theta = \sigma_1 + \sigma_2 + \sigma_3$, and is sometimes called the bulk stress. In a triaxial test, $\theta = \sigma_1 + 2\sigma_3$, where σ_1 is the axial stress and σ_3 is the confining pressure. Figure 4-5 shows typical data from a recent review [4-11] of roadbed material properties. These results could be approximated by

$$E_R = K\,\theta^n, \tag{4-1}$$

where K is a material constant, and n defines the slope of the line on a log-log plot. These data show that material type (gravel versus crushed stone) is not a major factor affecting the resilient response of granular materials.

The stress dependent properties of fine grained soils typically found in track subgrades are considerably different from those of granular materials used for ballast and subballast. Generally, the resilient modulus of fine grained soils decreases when the deviatoric stress is increased, see Figure 4-6. The resilient modulus is almost constant for high values of deviatoric stress.

4.2.4 Ballast and Subgrade Material Settlement Properties

The permanent, or plastic, deformation of roadbed materials from cyclic loading is the governing parameter for settlement. Barksdale [4-14] has shown that plastic strain depends on the deviator stress, $\sigma_D = \sigma_1 - \sigma_3$, the confining pressure σ_3, the number of load applications N, the aggregate type and gradation, the density, and the degree of saturation. Figure 4-7 shows some typical results wherein the cumulative deformation of a granite ballast material in a triaxial test is nearly proportional to the logarithm of the number of loading cycles. Similar data are reported [4-25] for limestone and slag materials for variations in density and gradation. Applied stress is the most important influence on plastic strain and these ballast materials do exhibit a threshold stress condition above which the strain increases very rapidly. Both repeated deviator stress and confining pressure affect the response, but the amplitude of the deviator stress is the most important factor. The stress ratio does not describe this behavior.

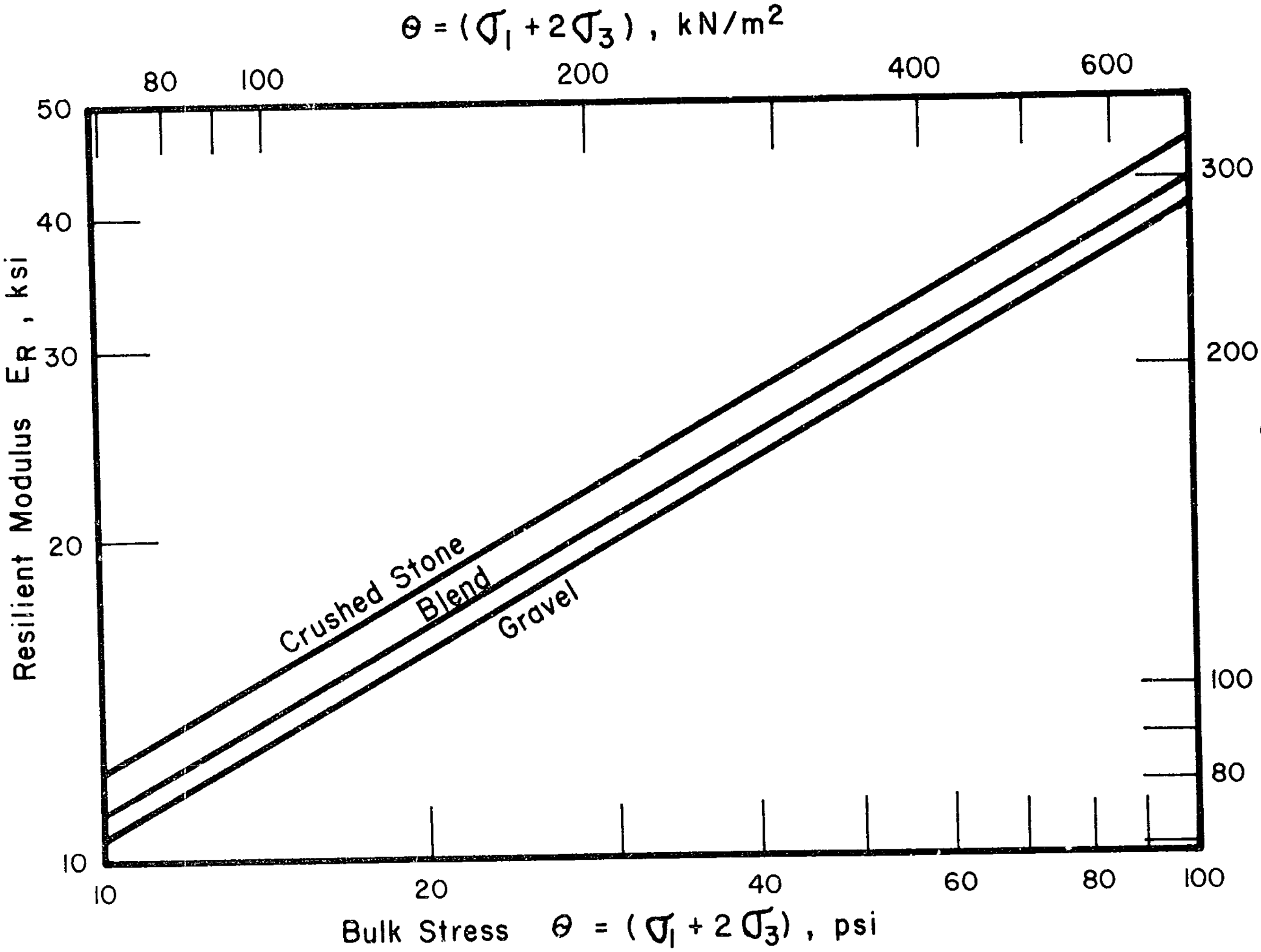


FIGURE 4-5. EFFECT OF MATERIAL TYPE ON THE RELATIONSHIP BETWEEN RESILIENT MODULUS AND BULK STRESS ([4-11], [4-12])

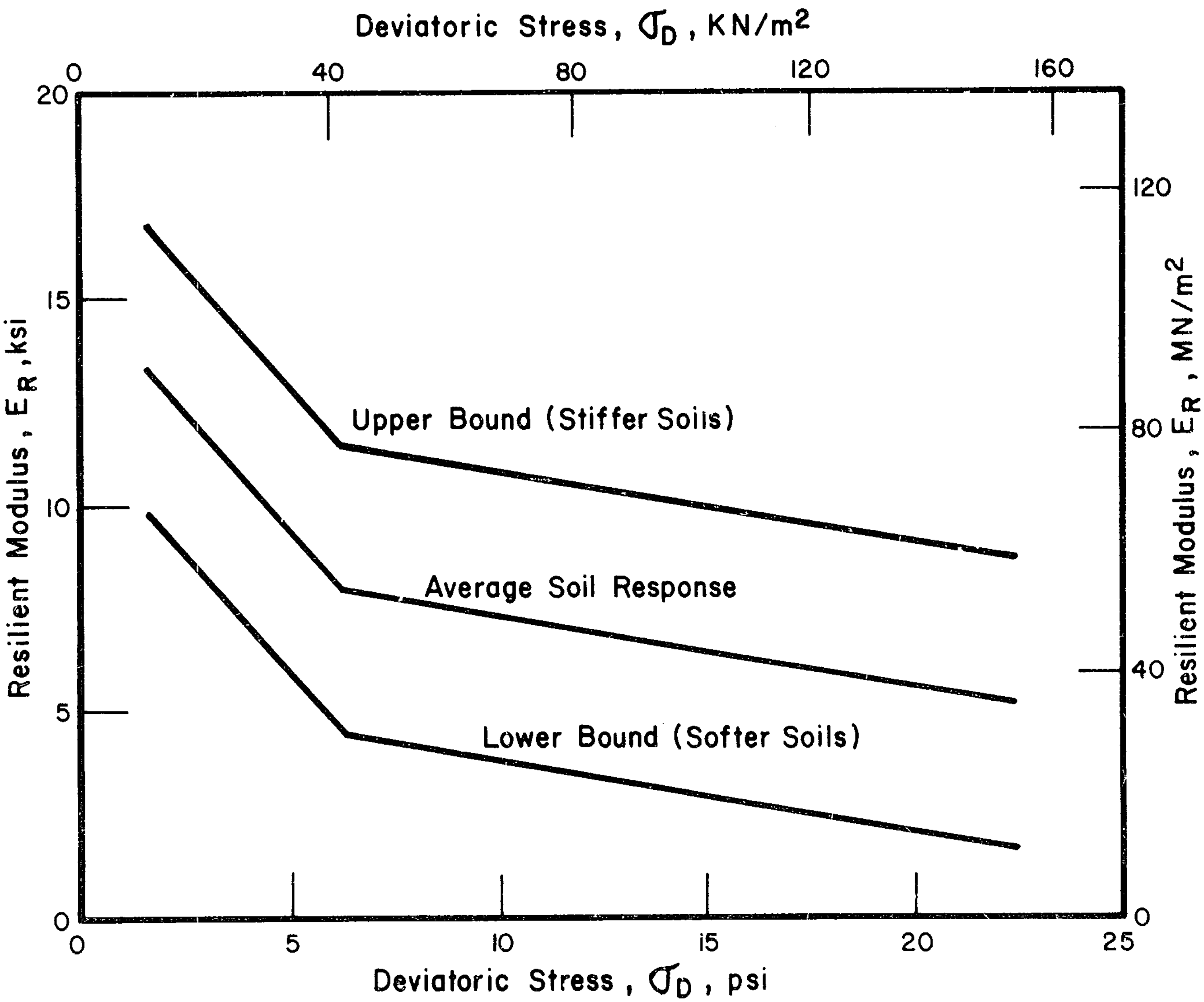


FIGURE 4-6. TYPICAL VALUES OF RESILIENT MODULUS FOR FINE-GRAINED SOILS [4-13]

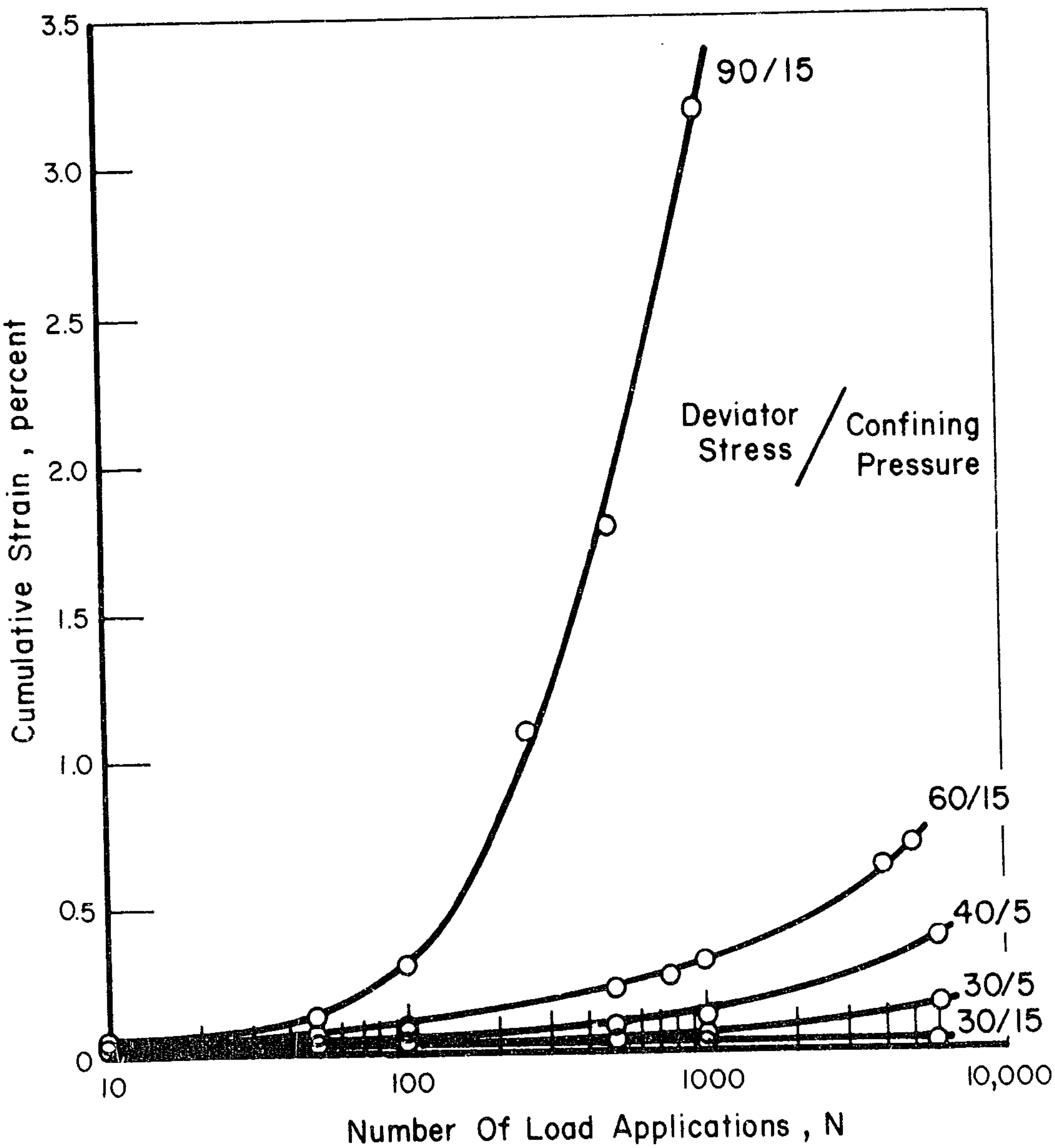


FIGURE 4-7. EFFECT OF STRESS LEVEL ON PLASTIC STRAIN RESPONSE OF NO. 4 GRADATION GRANITIC GNEISS BALLAST, HIGH DENSITY [4-25]

An analytical representation of the permanent deformation of ballast type materials in traiaxial tests is given by Robnett, et al. [4-11] from ORE results [4-18] as

$$\varepsilon_p = 0.082\ (100\ n - 32.8)\ (\sigma_1 - \sigma_3)^2\ (1 + 0.2 \log N) \tag{4-2}$$

where ε_p = permanent axial strain after N loading cycles

n = initial porosity

$\sigma_1 - \sigma_3$ = deviator stress, kgf/cm^2

N = number of repeated loading cycles.

This relation shows that permanent deformation is proportional to the deviator stress squared, which emphasizes the role of heavy cars in track deterioration. This type of model has also been evaluated in [4-25].

Barksdale [4-19] has also developed data for granular materials. An equation in the following form appears to represent test data for plastic strain for a specified number of load applications, N_o:

$$\varepsilon_p = \frac{(\sigma_1 - \sigma_3)\ /\ (K\ \sigma_3{}^n)}{1 - \dfrac{(\sigma_1 - \sigma_3)\ R_f\ (1 - \sin\varphi)}{2\ (C\ \cos\varphi + \sigma_3\ \sin\varphi)}} \left(\frac{N}{N_o}\right)^m \tag{4-3}$$

where ε_p = plastic axial strain

$K\sigma_3{}^n$ = relationship defining the initial tangent modulus as a function of confining pressure, σ_3 (K and n are constants), psi

C = cohesion, psi

φ = angle of internal friction

R_f = a constant relating compressive strength to an asymptotic stress difference, $0.75 \leq R_f \leq 1$.

Repeated load triaxial tests also provide useful data for cohesive subgrade soils. Figure 4-8 shows some typical results for the permanent strain of a subgrade material as a function of deviatoric stress, σ_D. This is identical to the applied axial stress for these tests because the confining pressure $\sigma_3 = 0$. The rapid increase in settlement with an increase in stress amplitude is evident. There also appears to be a threshold stress level above which

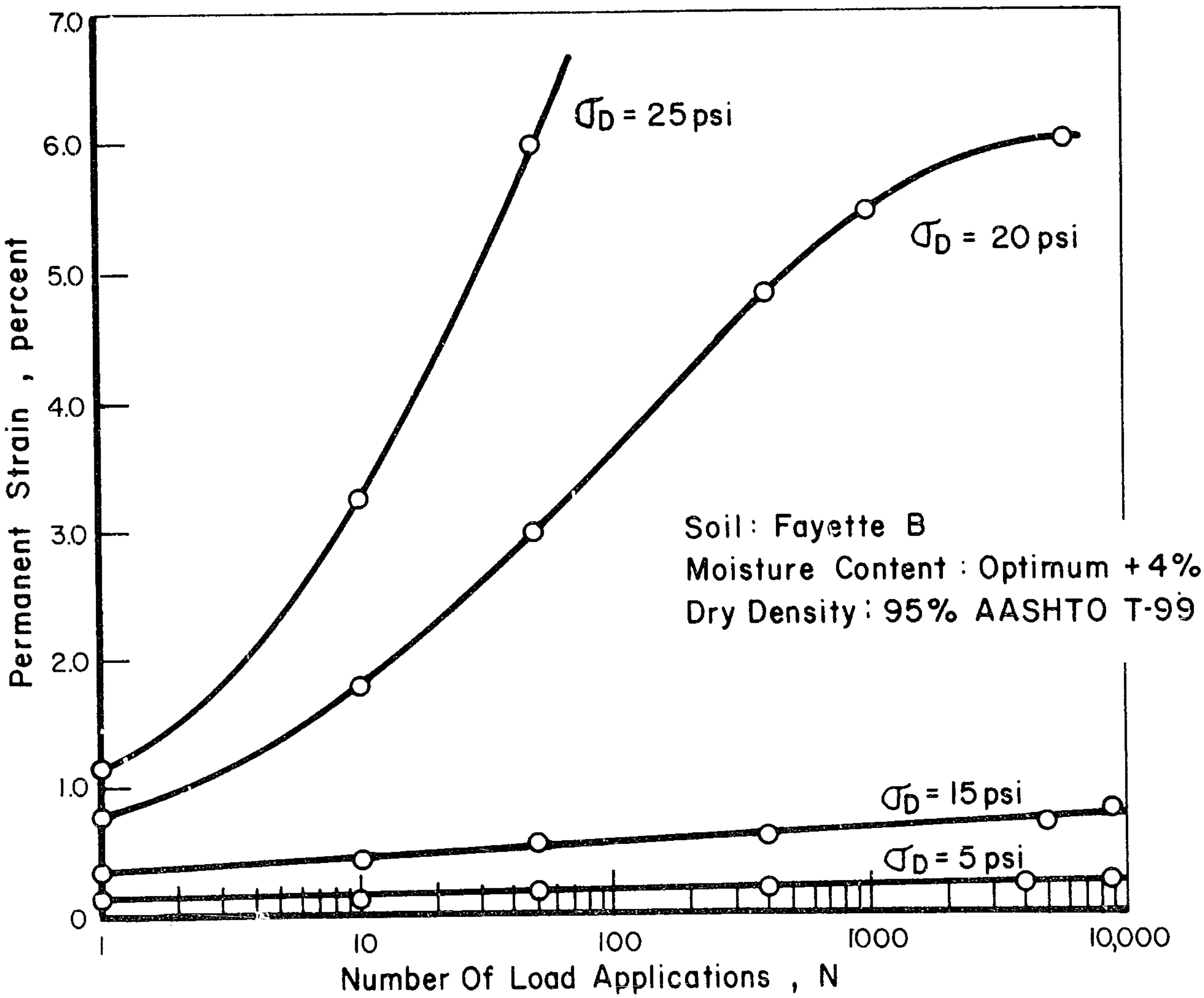


FIGURE 4-8. TYPICAL PERMANENT STRAIN BEHAVIOR FOR SUBGRADE SOIL SUBJECTED TO REPEATED LOADS [4-25]

the settlement rate is quite rapid and below which the settlement tends to stabilize. Figure 4-9 shows this behavior for three different moisture-density conditions. These data are based on the cumulative strain after 5,000 load applications.

Data from repeated load tests of several subgrade materials are reported in [4-25]. Several of these are listed in Table 4-4 where soil classification was used as a reference value to obtain a general rating and range of bearing values. These can be found in numerous references, such as [4-26]. The data in [4-25] were reviewed to obtain values of deviator stress which produce a specified value of permanent strain (0.5% at 5,000 cycles) for comparison purposes and an estimated threshold stress. Much of the data from repeated load tests do not correlate very well with standard bearing values. For example, the Drummer B soil with the highest repeated load strengths would be given the lowest bearing value based on soil classification. The large effect of increased moisture is also evident. Data of this type show the need for performing repeated load tests on specific subgrade materials which will be used for railroad track.

For computational purposes, an equation of the form

$$\varepsilon_p = AN^b \qquad (4\text{-}3)$$

where ε_p = permanent strain

N = number of load applications

A,B = experimentally determined coefficients

can be used to represent repeated load test data fairly well [4-25]. Correlation tests show that log A correlates significantly (at the 95 percent confidence level) with the deviator stress amplitude, but an analytical expression for this dependence is preferable.

An elastic layered analysis of the track structure like the MULTA model can be used to estimate total settlement as a function of traffic loading. The principal elastic stresses and the deviator stress would be predicted for each layer of ballast and subgrade. Data from repeated load triaxial tests similar to those discussed previously would be used to relate plastic strain for the predicted stresses and the number of load cycles.

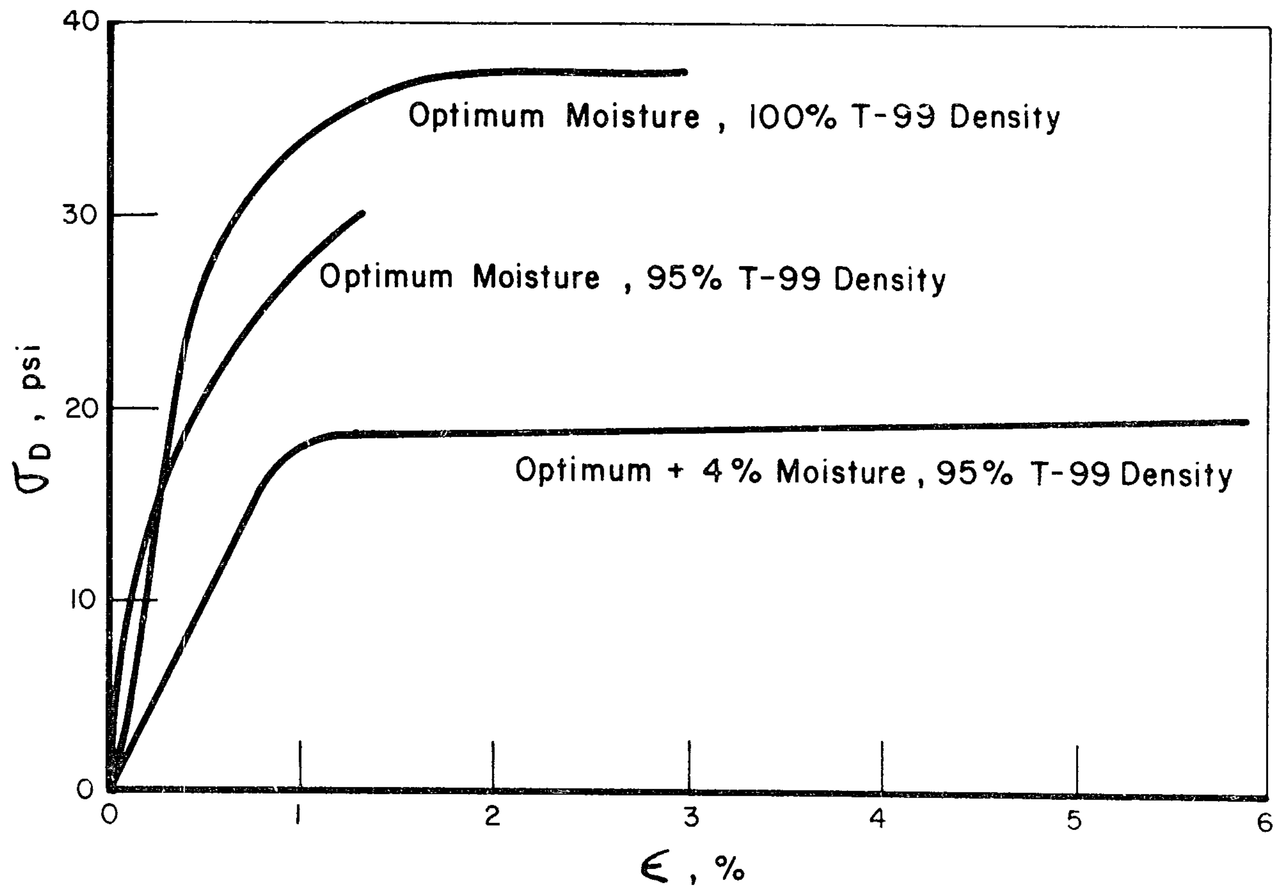


FIGURE 4-9. DEVIATOR STRESS-PERMANENT STRAIN RELATION AT 5000 LOAD APPLICATIONS FOR FAYETTE B [4-25]

TABLE 4-4. REPEATED LOAD PERFORMANCE DATA FOR SEVERAL SUBGRADE MATERIALS

Pedalogical Name & Horizon	Parent Material	Unified Soil Classification	General Rating	Bearing Value, psi	Approx. Repeated Load Threshold Dev. Stress, psi: 100% T-99 Den. Opt. Moist.	95% T-99 Den. Opt. Moist.	95% T-99 Den. Opt. + 4% Moist.
Davidson B_2	Residium developed over basic igneous and metamorphic rock	MH	Poor Subgrade	10–18	32–35	22–25	20–25
Dickinson C	Fine sandy loam	SM	Good to Excellent Subgrade	20–45	3.8–10	2.9–5	0.7–5
Drummer B	3-5 Ft. of loess on Wisconsin Till	CH	Very Poor Subgrade	8–12	50–60	NA–50	40–40
Fayette B	> 4.5 ft. of Peorian Loess	CL	Poor to Fair Subgrade	12–25	28–30	22–30	10–18
Fayette C	> 4.5 ft. of Peorian Loess	ML	Poor to Fair Subgrade	12–25	12–37	14–25	3–16
Norfolk B_2	Residium from thick beds of unconsolidated sandy loams and sandy clays of the coastal plain	SC	Fair to Good Subgrade	20–30	17–30	15–25	2–10

Note: Lower limit of range based on stress level at 0.5% permanent strain for 5,000 cycles. Upper limit based on estimated threshold stress.

The total plastic deformation in the layered structure can then be obtained by summing the deflections in each layer of thickness h by

$$\delta^p = \sum_i \varepsilon_i h_i . \qquad (4-5)$$

The MULTA program can be used to divide the roadbed structure into multiple layers to provide improved estimates of the average strain where stress gradients are significant.

4.2.5 JNR Track Settlement Criteria

The Japanese National Railways (JNR) has also done considerable research on the long-term degradation of track. Results from tests using a rotating mass shaker (vibrosir) have led JNR to conclude that tie settlement under dynamic loading is mostly affected by ballast accelerations, and that the amount of settlement can be reduced by increasing the ballast or subballast depth [4-20]. It should be noted, however, that some BR results indicate that increasing ballast depth can actually increase track settlement. The role of ballast depth in track performance is not clearly defined.

Figure 4-10 shows a typical example of the relation between tie settlement and number of loading cycles. The initial settlement rate is quite high, but it gradually decreases to a relatively constant rate. An analytical relation for this from [4-20] is

$$y = C_1 - C_2\, e^{-\alpha N} + \beta N, \qquad (4-6)$$

where C_1 = the total settlement from ballast compaction

N = number of loading cycles

$C_2\, e^{-\alpha N}$ = residual void after N loading cycles

$C_1 - C_2$ = amount of free play in ballast and other track components

βN = tie settlement due to ballast flow and long-term degradation.

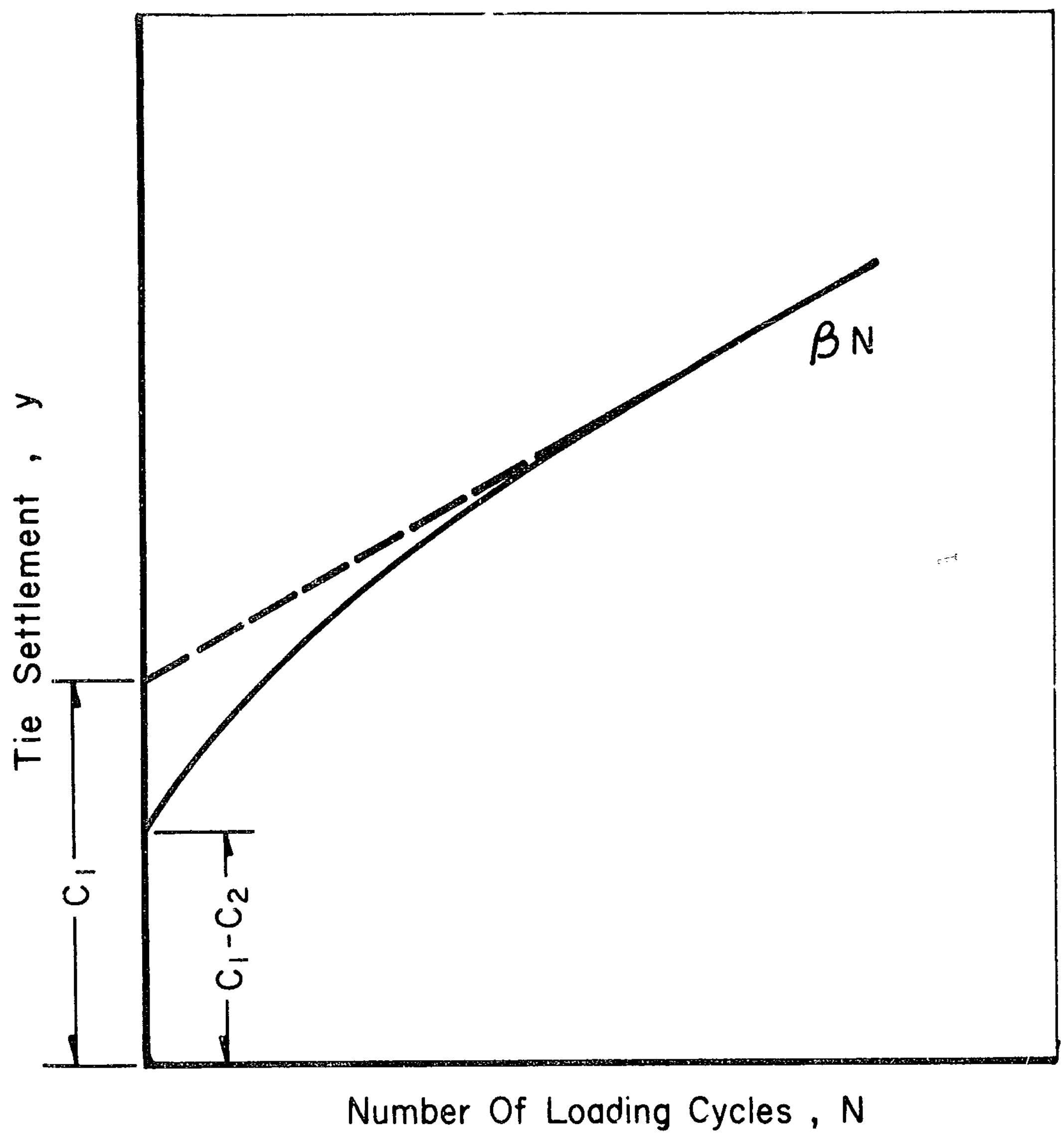


FIGURE 4-10. TIE SETTLEMENT UNDER REPEATED LOAD

With no free play in the track, $C_1 - C_2 = 0$, and Equation 4-6 can be reduced to the form

$$y = C_1 \ (1 - 3^{-\alpha N}) + \beta N \ . \qquad (4\text{-}7)$$

This shows that settlement is proportional to N, whereas most other material and track degradation data show a log N relation for long-term settlement.

The JNR results indicate that vibratory loads enlarge the region of ballast where compaction occurs because the vibration reduces the ballast shear resistance. Therefore, JNR suggest that C_1 is proportional to ballast acceleration. However, the initial track settlement is not of major importance because it can be reduced considerably by compacting equipment.

The JNR results also show that ballast flow, as measured by β, increases with the pressure at the tie/ballast interface and tie acceleration. Vibratory tests on the track produce both high- and low-frequency accelerations. While the high-frequency components can be much larger than the low-frequency, only the low-frequency acceleration corresponding to the excitation frequency from the vibrosir (about 16 Hz) was used for correlating settlement measurements. Increasing ballast depth increased the overall track stiffness and reduced tie displacements and accelerations for a constant load. Typical results for tie acceleration during settlement tests range from 2.2 g with 10 in. (25 cm) of ballast to 1.2 g with 20 in. (50 cm) of ballast. Acceleration at various depths in the ballast decreases in accordance with the local deflection. Accelerations about 4 in. (10 cm) under the tie bottom are about 40 percent lower than the tie acceleration, and it is this acceleration that is used by JNR to reference their settlement data. A typical empirical relation for tie settlement rate is

$$\beta = 1.10 \ (P_a \ A_b - 0.4) \qquad (4\text{-}8)$$

where β = settlement rate per cycle (0.001 mm)

P_a = average tie bearing pressure (kg/cm^2)

A_b = ballast acceleration 10 cm under the tie bottom (g).

It is recommended [4-20] that this relation for β is only valid in the range of $P_a A_b < 1.8$, because β increases very rapidly when this limit is exceeded. For a typical value of average ballast pressure of 40 psi (2.8 kg/cm^2), this implies a ballast acceleration less than 0.64 g and a settlement rate of β = 154 mm (6 in.) per million load cycles (axles). For a typical U.S. freight traffic, this would represent a settlement of 0.38 in. per MGT of traffic--an unreasonably high value in consolidated track. In any case, the parameter P_aA_b may be useful index for vertical track settlement to include the combined effects of ballast pressure and acceleration, where the acceleration is the low-frequency component due to wheel passage which neglects the higher frequency components due to impact between the tie and ballast. It is also interesting to note that since both acceleration and ballast pressure increase with axle load, total deformation will depend on a power relation for axle load. This agrees with results from other investigations. The use of the acceleration term, however, combines a track deflection and vehicle speed dependence.

Measurements of ballast settlement from vibration have also been reported by Raymond [4-21] based on laboratory tests at Queen's Univerity. These measurements used a container of ballast about 10 in. high and 10 in. in diameter, and the container was placed on a vibration table. Vertical excitation at constant acceleration amplitude showed that varying the frequency over the range of 10 - 50 Hz did not affect settlement. Acceleration amplitude most affected settlement, with settlement (density) increasing rapidly for peak accelerations above about 1.25 g and then leveling off for peak accelerations above 2 g, see Figure 4-11. This result is not unexpected for a laboratory test where vibration in excess of 1.0 g will float the ballast against normal gravity loads, but these results cannot be related directly to track where the acceleration amplitude diminishes rapidly with depth. However, it appears that accelerations greater than 1.0 g at the tie/ballast interface may cause a substantial increase in settlement rate, and this does correlate rather well with the limits from the JNR track tests.

4.2.6 The European Approach to Track Settlement

Prud'homme [4-22] suggests that the average vertical track settlement is proportional to traffic tonnage, at least on the track in Europe where the average axle load is not too high. In the United States, where axle loads are much higher than in Europe, results indicate substantial increase in surface maintenance results from relatively small increases in axle loads even though tonnage may be constant. The average settlement rate reported in [4-22] for Europe varies from 2 to 10 mm per factor of 10 increase in traffic tonnage when the track is in good condition. However, Figure 4-12 shows that the

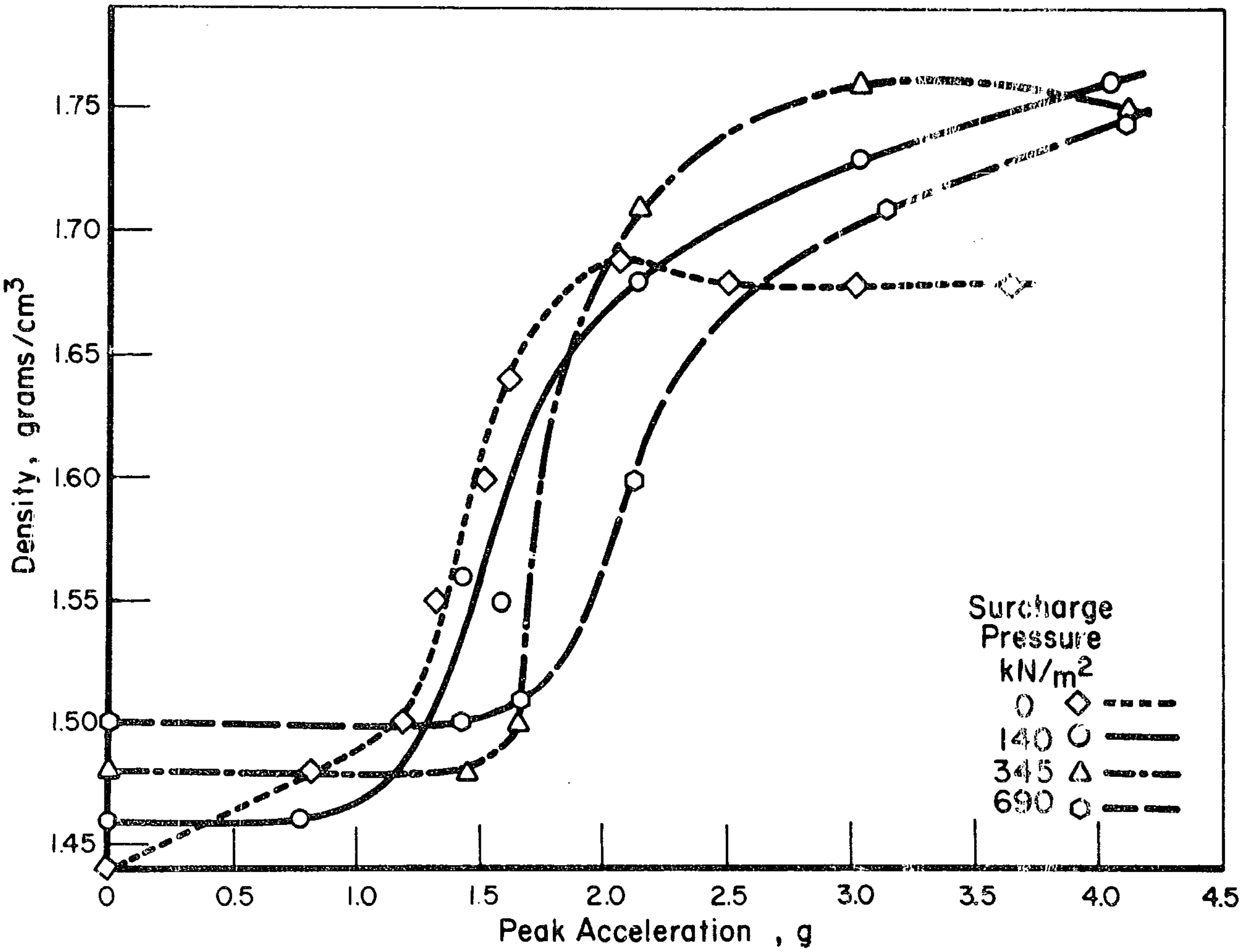


FIGURE 4-11. EFFECT OF ACCELERATION ON BALLAST SETTLEMENT IN LABORATORY TESTS [4-21]

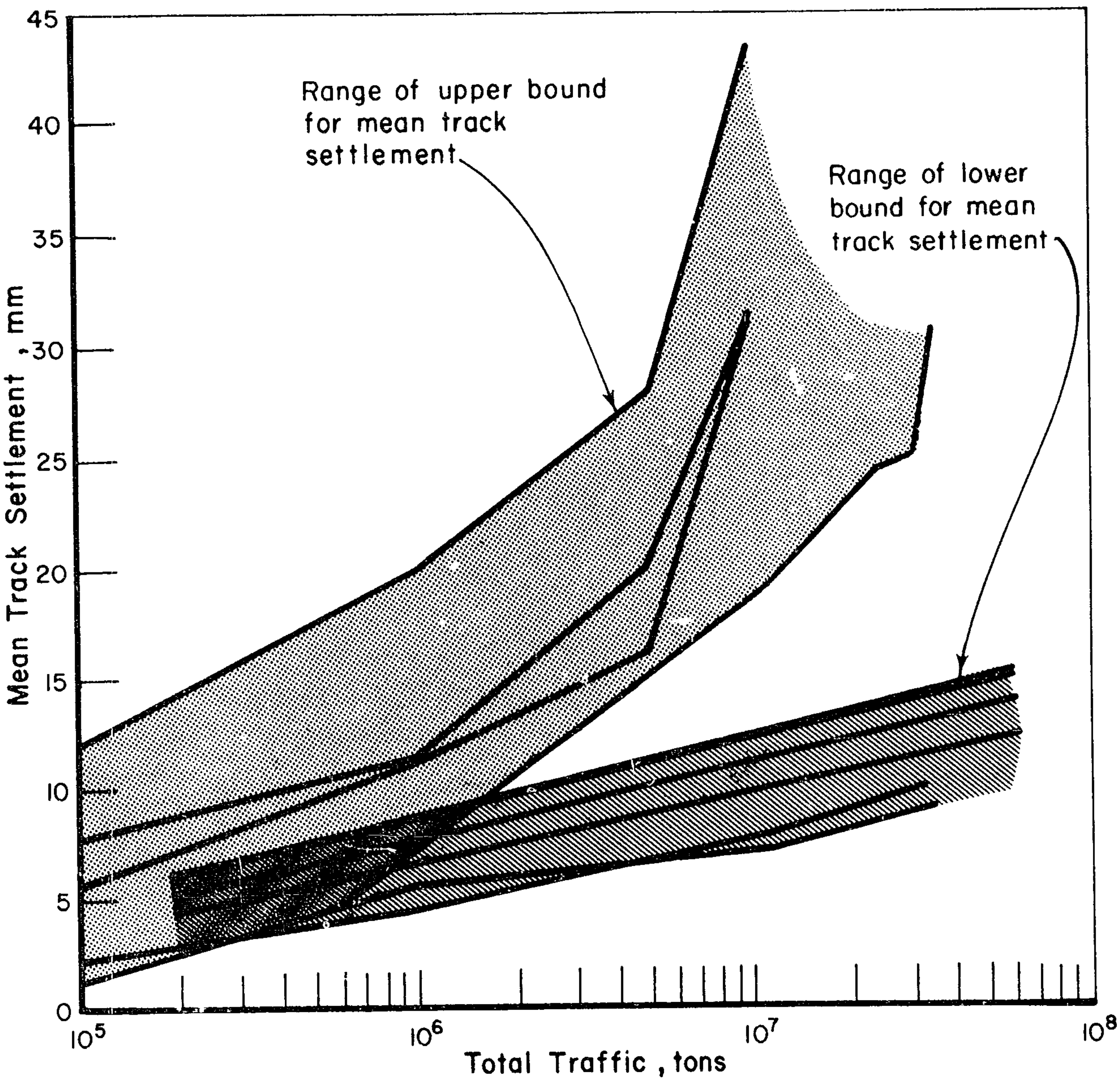


FIGURE 4-12. TYPICAL SETTLEMENT DATA FOR EUROPEAN TRACK [4-22]

settlement rate increases rapidly for some track when total traffic has exceeded 50 - 100 MGT.

With regard to differential settlement, European results shown in [4-22] indicate that in the short wavelength range below 24-30 feet, the track deforms most rapidly at discrete wavelengths which match the vertical frequency of the vehicle sprung mass and the operating speed (e.g., 3 Hz at 60 mph generates a 29-foot wavelength). This is particularly important for unit trains where all vehicles are identical and speeds are constant. At longer wavelengths, the differential settlement is due more to the spatial variations in track conditions, and spectral peaks in differential settlement are not too significant. An investigation [4-22] of the transfer function for a tamper indicates that a tamper is quite effective for removing track geometry defects in the wavelength range of 15-75 feet, but it may actually increase the geometric defect for long wavelengths. This is particularly bad for a very high speed passenger train where a 1 Hz natural frequency at 150 mph will be excited by geometric defects at a 200-ft wavelength. It is also generally recognized that disturbing the ballast by tamping restarts the track degradation cycle wherein the permanent deformation is initially quite rapid until the track is reconsolidated. There is a real need to be able to level track without disturbing the consolidated ballast.

Work by the ORE on Question D117 [4-23] is directed toward developing a model to relate the capital cost of conventional track to the subsequent maintenance costs for present and future traffic, and thereby designing track for minimum life cycle costs for any specified operating condition. The information needed to formulate track maintenance limits based on safety, passenger comfort, and the economics of maintaining track at sufficient intervals to prevent rapid deterioration are of specific concern and are not now available.

One objective of the ORE work is to determine how track geometry varies with time and traffic and what parameters govern this change. In their planning, they have proposed the use of a "viability" index to represent the degradation with time of a particular track geometry parameter (e.g., profile, line, gage, cross-level, twist) for a well defined track layout, design, and specific traffic. Therefore, the time required for the viability index to reach a limiting value becomes the maintenance interval, and this maintenance interval can be adjusted by changing track design or train operations, to obtain an optimum cost solution within the constraints of operating safety or passenger comfort.

In order to determine the effect of traffic on the viability index, the ORE has suggested using a notation from AASHO for highways. They convert actual traffic to equivalent theoretical traffic T_f (gross tons per day) having a number of N_f of reference axleloads P_o. This theoretical traffic would require identical maintenance intervals on a track with permissible speed V_o as the actual traffic would on the actual track which is classified for operation at speed V_{max}. A form suggested by ORE [4-23] for this relation is

$$T_f = N_f P_o = \left[\frac{V_{max}}{V_o}\right]^{1/\beta} \sum_i T_i \left[\frac{P_i}{P_o}\right]^{\alpha-1} \left[1 + \frac{\alpha\ (\alpha-1)}{2}\ \frac{\sigma_i^2}{P_i^2}\right] \quad (4\text{-}9)$$

where $T_i = N_i P_i$ gives the fraction of tonnage having a mean axle load P_i and standard deviation σ_i. The standard deviation is intended to include variations of nominal static wheel loads plus dynamic effects of particular vehicles operating at speed V_i over a particular track. This type of data can be obtained from a statistical analysis of track load measurements from a revenue service. The data would be analyzed in several car weight categories to give the detailed breakdown on P_i and σ_i.

The α and β are empirical parameters which depend on the track design and layout, and vehicle characteristics. A large number of tests would be required to identify these parameters for an equivalent theoretical traffic. However, the philosophy behind this approach is useful in that it suggests a framework for correlating track degradation results for different track and traffic conditions. The disadvantage is that empirical results cannot be extrapolated to new track configurations where test results are not available.

4.2.7 Current U.S. Track Geometry Specifications

The Federal Railroad Administration (FRA) track safety standards [4-24] are the current U.S. specifications on track geometry, except standards which may be used by individual railroads. The FRA standards are intended to be maximum allowable deviations for safe operation at a specified speed for each track class. Consequently, more refined criteria related to comfort and loading damage are needed to implement a performance index methodology. The rapid transit industry has a similar set of geometry standards which are somewhat more restrictive for the same operating speeds because of the emphasis on passenger comfort rather than safety.

The FRA track safety standards specify maximum midpoint deviations in the profile of each rail under a 62-ft chord, so the geometric characteristics as a function of wavelength are important for a performance index using these criteria. Similar limits are given for the deviations in cross level on tangent and spirals and the difference in cross level (twist) in intervals less than 62 ft on tangent and spirals. Specific dimensional limits are specified for each track class, which also designates a different maximum operating speed for freight and passenger traffic.

4.2.8 Summary of Track Surface Deterioration Criteria

A review of the literature indicates that there are no quantitative performance measures to relate a description of the railroad traffic with the track design parameters and track surface deterioration rate. Some laboratory investigations using triaxial repeated load tests with granular and cohesive soils give an indication of how typical ballast and subgrade materials will behave under uniform loading conditions. Settlement rate appears to be proportional to some power of deviatoric stress, σ_D^N, and the settlement increases proportional to N or log N, where N is the number of cycles at a specified loading. A cumulative settlement law for combining the various stress amplitudes and number of cycles representing typical traffic has not yet been established for utilizing results from these laboratory material tests.

The JNR has done some track settlement tests to develop empirical relations for settlement due to ballast flow and long-term degradation. These results indicate a settlement rate that is related to the product of average tie/ballast pressure and ballast acceleration. A linear dependence on a number of loading cycles is proposed for long-term degradation, following an initial high rate of settlement before consolidation has been established. The linear relation is particularly attractive for combining traffic conditions with different axle loads and train speeds. The JNR made no attempt to separate ballast settlement from subgrade settlement except to determine the effect of different ballast depths on overall settlement rates.

The current state of the art regarding track settlement indicates that only a relatively simplified performance index is justified. However, this index should include the fundamental ballast and subgrade parameters needed for evaluating the effects of variations in track design parameters. This requires identifying the relative contributions from the ballast and subgrade to the total settlement. One possible format for a Track Settlement Index (TSI) is based on total vertical settlement per MGT of a defined traffic. This could be represented as

$$TSI = \frac{\sum_i \beta_i N_i}{\sum_i N_i P_i} ,$$

where $N_i P_i$ gives the tonnage for N_i number of axles at mean axle load P_i, and β_i is the track settlement rate for those N_i axles.

The track settlement rate is a result of the cumulative settlement in each of j layers representing the ballast and subgrade,

$$\beta_i = \sum_j C_j \sigma_{d_{ij}}^{\alpha_j} h_j .$$

The settlement in each layer depends on the permanent strain per cycle given by $C_j \sigma_{d_{ij}}^{\alpha_j}$ and the layer thickness h_j, where

$\sigma_{d_{ij}}$ = deviatoric stress in the j layer for the traffic of P_i, N_i

C_j and α_j = empirical parameters for the particular material and stress condition of layer j.

The critical parameters which a track analysis model must provide are the average deviatoric stres in a layered representation of the roadbed for the statistical loading description of the railroad traffic. Other operating parameters which affect dynamic wheel loads, such as train speed, track roughness, etc., would be included by using a probability density description for wheel/rail loads to calculate the resulting roadbed stresses.

The TSI representing the rate of uniform vertical settlement can be used as the basis for estimating differential vertical settlement as a function of wavelength by using the relations similar to those in Figure 4-4. A separate relationship for rail profile and track cross level would permit a prediction of surface deterioration to be compared with the maximum allowable deviations under a 62-ft chord that are currently the basis for track safety standards.

Figure 4-13 shows a flow chart of the major elements in an overall track surface deterioration model. Most of these elements have been discussed in detail, but the flow chart emphasizes the interactions between the track response and settlement models and the statistical nature of track loads and material properties. Track maintenance is included as a key end item in the deterioration model, because the track can be restored to essentially new condition as required. The track deterioration and maintenance strategies can also be combined with a cost model to evaluate optimum track design and maintenance strategies in terms of minimum maintenance cost or overall life-cycle costs.

In the initial stage of implementing the track surface deterioration model, the traffic response is obtained using MULTA for static wheel/rail loads to evaluate a range of track design configurations. These track response parameters in terms of stresses and strains at any point in the layered roadbed can then be used as input to a set of transfer functions for ballast and subgrade settlement. The statistical nature of track loads and material properties would be introduced at this stage to provide probabilistic estimates for the mean uniform track settlement as a function of time and traffic. The final step in the process of predicting spatial variations in track geometry is to relate the mean track settlement to the expected spatial variations as a function of wavelength. This can then be related to the physical measurements of track geometry obtained by survey or a measurement car, and can be evaluated relative to safety and serviceability criteria such as the FRA track safety standards. A feedback loop has been included to show the dependence of track loads on the current track geometry.

The statistical nature of wheel/rail loads and the distribution of these loads to the track components has been discussed in the previous interim report [4-1]. Track response parameters for several variations in track design configuration have been presented in Section 3 of this report.

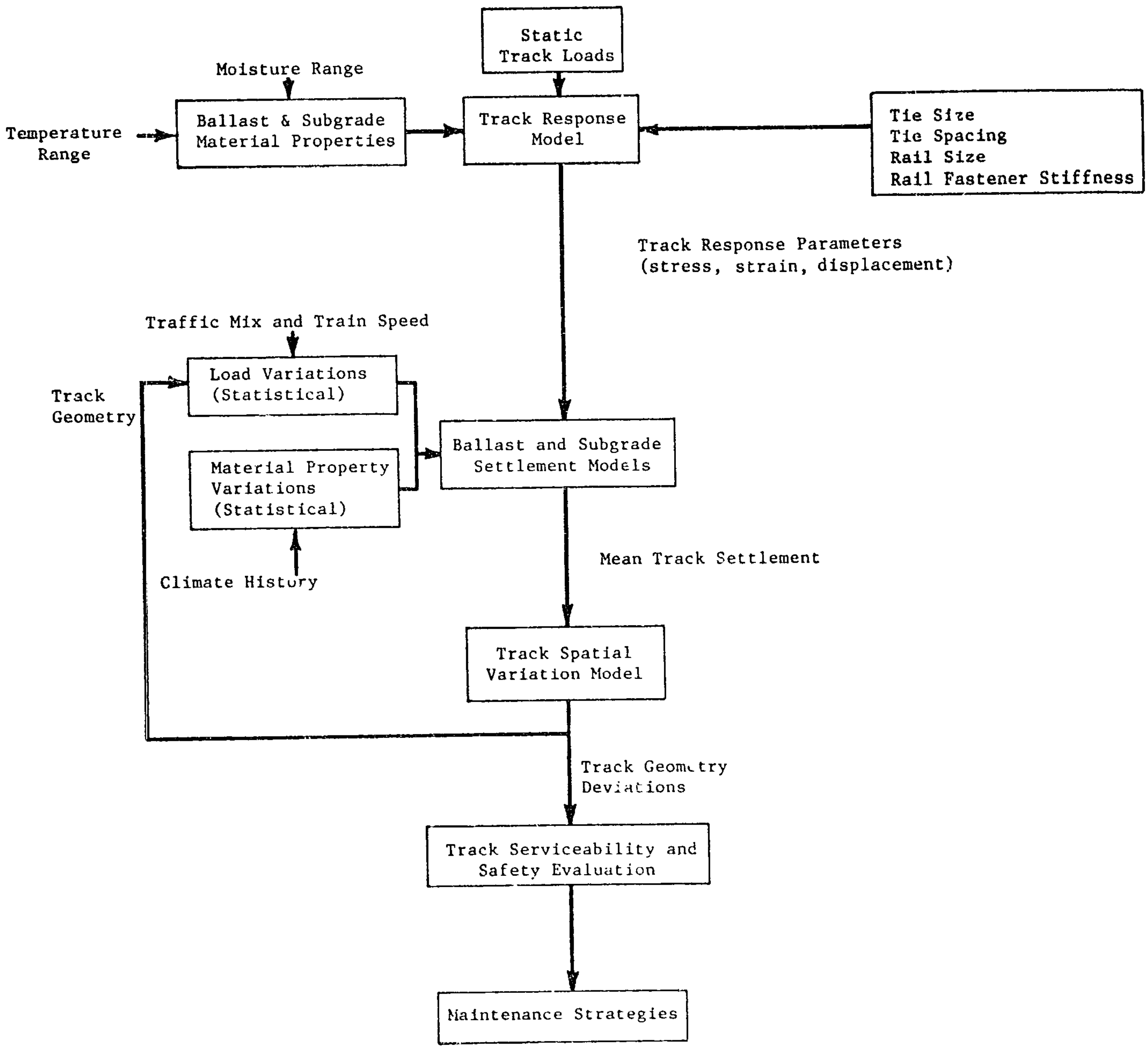


FIGURE 4-13. FLOW CHART FOR TRACK SURFACE DETERIORATION MODEL

5. TRACK LATERAL STRENGTH

Track design procedures usually result in selecting tie size, tie spacing, and rail size based on the response to vertical loads, as discussed in Section 3. Therefore, the lateral load requirements have relatively little influence on the final track configuration except for the fastener requirements and the width of the ballast shoulder at the tie ends.

Track lateral strength is an important factor in maintaining track alinement under continuous traffic and for the safety aspects of train derailments. The lateral strength of unoccupied track must be sufficient to prevent track shifting from rail thermal loads. The resistance to the lateral component of the rail thermal forces is provided primarily by the interaction between the ballast and ties. However, a rail fastener which restrains the rail from rotating in a horizontal plane (rotation about a vertical axis) can significantly increase the track's lateral strength. This represents an important fastener design parameter that has not been used fully in hardware currently available.

When a track is occupied by a train, the lateral strength must be sufficient to resist both the thermal loads and the lateral component of the wheel loads. The presence of the vertical wheel loads is an important factor in increasing the effective lateral track strength for these conditions. The track design parameters which affect lateral strength are discussed in this section.

5.1 LATERAL RESISTANCE OF UNOCCUPIED TRACK

The mechanics of lateral track loading are quite complex, and this review is limited to a discussion of the type of information that is being used for track design. Considerable research on lateral track characteristics has been done in Europe and some measurements of tie and track lateral resistance have recently been made in the U.S. But, there is still much less reliable information about lateral track characteristics than about vertical characteristics.

The lateral resistance provided by a tie in ballast is a principal factor for lateral strength. The available data show considerable variation. In Europe [5-1], values of the ultimate lateral resistance for single wood ties in 1.2 to 2.5-in. size ballast were between 720 lb and 1150 lb per tie with freshly laid track. After several months of traffic, the lateral resistance often rose by 50 percent or more. Monolithic concrete ties gave results approximately 15 percent greater than wood ties. [5-1]

Tests have also been made in Great Britain to find the effect on lateral resistance of ballast shoulders of different sizes and shapes for freshly-placed 1.5 to 2.0-in. crushed stone and slag ballast. Negligibly small improvements in resistance were obtained if the flat top of the shoulders exceeded 14 in., and there was only a little reduction in resistance when the flat top of the shoulder was 12 in. When the flat top was only 7 in., however, the resistance was 60 to 70 percent of that obtained when the flat top was 12 in. These data indicate that increasing the ballast shoulder width beyond about 12 to 14 in. is of little value, although some of the currently used track design equations assume a linear increase with no specified limits.

Some additional data on the lateral resistance of single ties (with rails disconnected) are listed on Table 5-1. Most of these measurements were made on French track [5-2], but some recent measurements [5-3] on U.S. track were also reported. These measurements were made in conjunction with a project to determine the effect of using a vibratory ballast consolidator on track strength following any type of track maintenance which disturbs the crib or shoulder ballast. The tie resistance data show considerable variation, as indicated by the wide range, but the mean resistance values for a total of 343 ties do show a 36 percent increase when the track is consolidated after maintenance. This advantage disappears after about 0.5 MGT of traffic.

Similar effects of traffic were evident from lateral resistance measurements of a complete track panel. There is some question whether lateral pull tests on single ties give an accurate measure of lateral track resistance, because of the difference in relative motion between ties. It is hypothesized that the lateral resistance on a single tie is increased artificially by the load transferred laterally through the ballast to adjacent ties. This can be avoided by connecting several ties together rigidly so that each tie undergoes the same displacement. Loading a track panel with the rails attached is

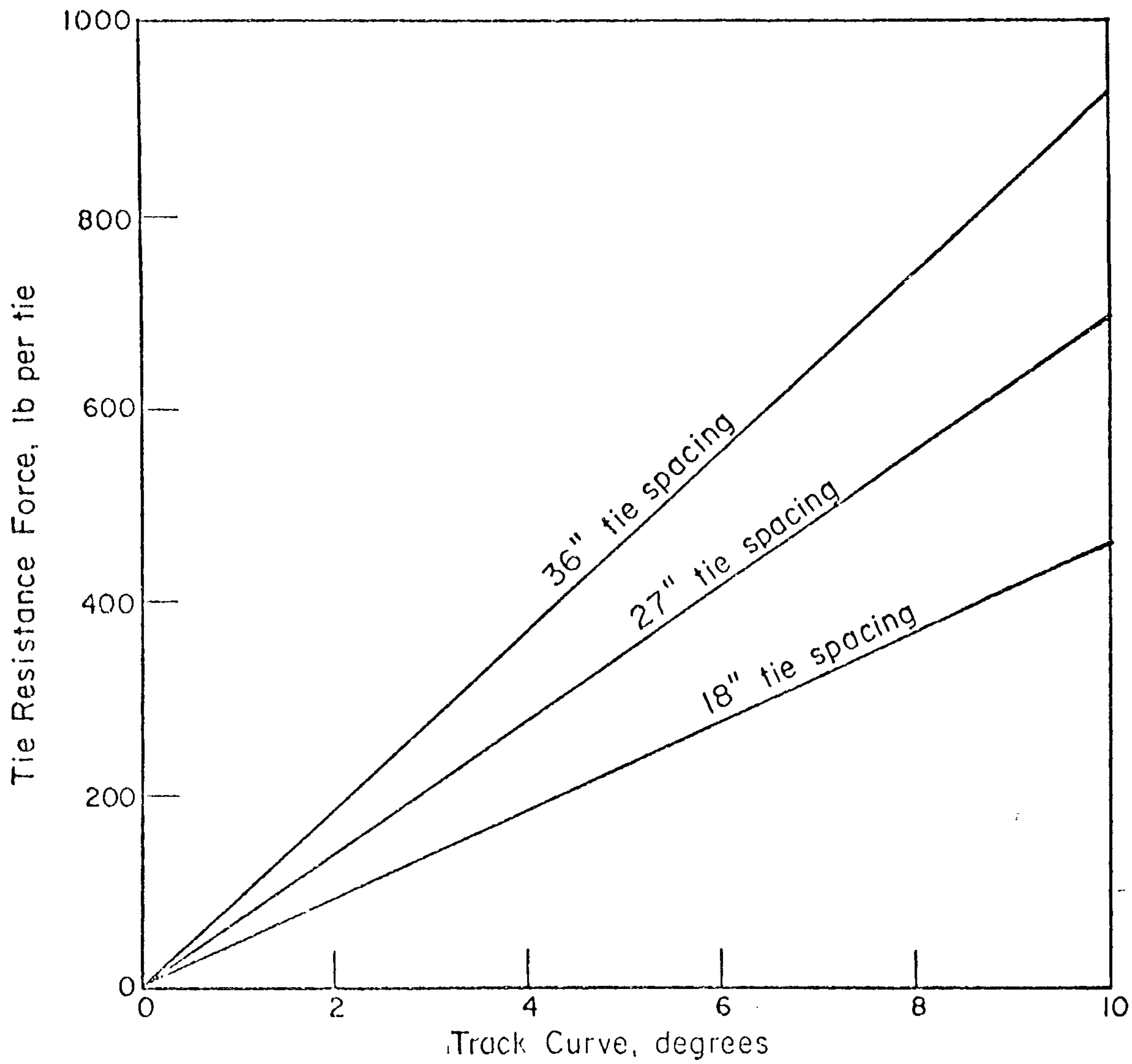


FIGURE 5-1. MINIMUM LATERAL TIE RESISTANCE REQUIRED FOR ΔT=70F RAIL TEMPERATURES

Some additional results from European tests show that a full 18-in. ballast shoulder contribures 20 to 30 percent of the tie resistance, the tie sides contribute about 20 to 30 percent, and the tie bottom is responsible for about 50 percent of the total resistance. The resistance from the tie bottom does depend on tie type (wood or concrete) and tie weight. The weight factor can be estimated using the data in the following section for critical lateral force versus vertical load. However, a conservative design approach is to estimate the lateral resistance by ignoring the contribution from the tie bottom. This represents the uplift condition from the precission wave in front and behind a railroad car truck. Although it has not been verified, this condition of zero ballast pressure may be the critical factor for alinement deterioration in the presence of high thermal loads from either high or low temperatures. In this case, lateral forces exist independently of the vehicle. A soft tie pad between the rail and tie or a fastener which permits free uplift of the rail will minimize this reduction in lateral resistance.

This discussion of the lateral resistance of unoccupied track summarizes conventional track design procedures which are based on preventing track shifting on curves during periods of elevated temperature. Buckling of tangent track is also of concern, but no reliable procedures for selecting the rail laying temperature or the track design have been available. Results from recent research by Kerr [5-6], however, now provide an improved method for selecting rail laying temperatures and evaluating track designs to avoid buckling on tangent track. This method shows that the critical temperature for track buckling depends on the rail size and the lateral or longitudinal resistance of the ties in the ballast. Results from parametric studies can be used to estimate the reduction in allowable temperature from track maintenance operations which disturb the ballast and reduce the tie resistance.

5.2 LATERAL RESISTANCE OF OCCUPIED TRACK

The net lateral load applied to the track by one axle of a car results from the flange force at one wheel and the two lateral components of the wheel/rail frictional force at both wheels on the axle. The ratio of this net lateral force to the total vertical axle load is generally considered to be a significant loading quantity governing the lateral shifting of track.

Koci and Marta [5-7] indicate that this ratio should be no greater than about 0.4 for safe operation with regard to lateral track shift on track in poor condition. Other data indicate that there is a considerable range in values for the resistance of track to lateral shift. This resistance depends on tie and ballast conditions and maintenance procedures.

The French [5-8] have made extensive measurements using a "Wagon Derailleur" car. This is a 2-axle railroad car having a special axle in the car center to apply variable vertical and lateral axle loads using air cylinders. The lateral load is applied by pressing a small roller against the back side of a standard wheel to force the wheel flange against the rail head. These tests start with a specified vertical axle load and make repeated runs over the test section with gradually increasing lateral loads. Measurements of track displacement showed that there was a threshold lateral load such that the track deformation increased rapidly for lateral loads which exceeded the threshold load.

Data from these tests showed that the critical ratios for track lateral shift range from a low of about 0.5 to as high as 1.25. Some of the other important conclusions from these tests are

a. The size of the ballast has little influence on lateral resistance
b. The resistance of the end-face of the tie is important. It can be improved further by increasing the depth of ballast above the tie base.
c. There is a distinct advantage offered by the additional end-face on 2-block concrete ties.
d. Retamping reduces the lateral resistance of the track structure.

Amans and Sauvage [5-9] present a detailed analysis of track lateral shift, including several equations which match experimental data from the derailleur car operated over a reference track. These equations are listed in Table 5-2 for convenience, and are shown graphically in Figure 5-2. Most of these results represent the lateral strength of track which has been maintained recently. It is generally recognized that any type of track maintenance which disturbs the ballast causes reduced lateral resistance until the track has been reconsolidated by traffic. The SNCF utilize data from track in this condition of reduced strength to establish a conservative limit for all train operations. An alternative approach would be to use track limits based on

TABLE 5-2. EQUATIONS RELATING TRACK LATERAL CRITICAL FORCE (H_c) TO VERTICAL AXLE LOAD (P)

Lateral Critical Force, Newtons		Description
1	$H_c = 0.3\ [P + 3.33(10^4)]$	New, unconsolidated track [5-9]. One axle pass for each value of H.
2	$H_c = 0.25\ [P + 4(10^4)]$	Same as (1), but with 10 axle passes for each H [5-9].
3	$H_c = 0.31\ [P + 4(10^4)]$	Old, but newly cleaned track [5-9].
4	$H_c = 0.615\ [P + 4(10^4)]$	Track consolidated by 9 MGT of traffic after cleaning [5-9].
5	$H_c = P/3 + 1(10^4)]$	SNCF reference track which has recently been leveled by ballast cleaning and tamping [5-9].
6	$H_c = 1.10\ [P/3 + 1(10^4)]$	Concrete ties increase lateral strength of reference track by 10 percent [5-9].
7	$H_c = \frac{P}{3} + 1.96\ (10^4)$	Concrete tie lateral strength given by Nouvion [5-10].
8	$H_c = 0.85\ [P/3 + 1(10^4)]$	SNCF lateral strength criteria for determining curve speed limits [5-9].

Note: Track is U33-46 kg rail with wood ties at 60 cm spacing except for (6) and (7) where concrete ties were used.

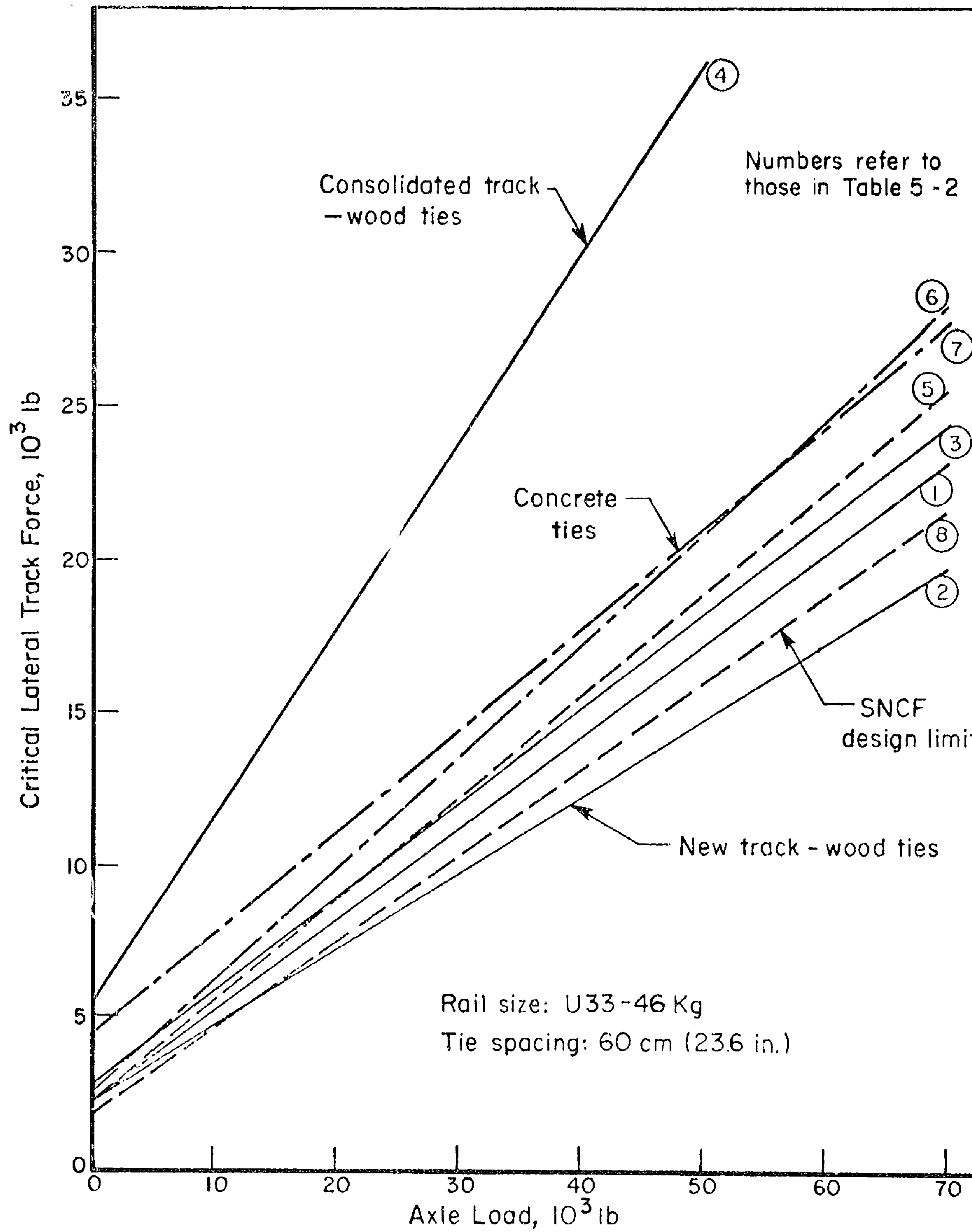


FIGURE 5-2. COMPARISON OF LATERAL TRACK FORCE LIMITS VERSUS AXLE LOAD

consolidated track for normal operation, and reduce train speeds on recently maintained track. However, this would require detailed information about how the track strength increases in service to insure safe operating speeds.

The equations listed in Table 5-2 are certainly useful for describing safety limits based on the measured lateral resistance of existing track, but they do not provide the information needed to assess the effect of track design parameters. However, Amans and Sauvage [5-9] have provided a more detailed empirical design equation to determine the critical lateral force as a function of the combined effects of axle load, radius of curvature, temperature, track modulus and rail stiffness. This equation is

$$H_c = \Gamma * H' \tag{5-3}$$

where H_c is the critical lateral threshold force for track shifting based on measured data for the resistance H'_c of a "reference track". The recommended value for this is listed as Item (5) in Table 5-2.

$$H'_c = 1(10^4) + \frac{P}{3} \text{ (in Newtons)}, \tag{5-4}$$

where P is the vertical axle load.

The dimensionless coefficient Γ representing all other influences normally falls in the range of 0.8 to 0.9. However, an empirical function has been developed to quantify the effects of temperature, curvature, track modulus, and rail stiffness. This expression is

$$\Gamma = \left[1 - \beta S \Delta\theta \left(1 + \frac{R_o}{R}\right)\right] \left(\frac{U}{U_o}\right)^{1/8} \left[\frac{\varepsilon (EI)^{1/4}}{(EJ)^{1/8}}\right] \tag{5-5}$$

where

R = curve radius, m

S = rail section area, m^2

$\Delta\theta$ = temperature change above neutral, ^oC

U = track modulus, N/m^2

EI = rail lateral bending stiffness, $N\text{-}m^2$

EJ = rail vertical bending stiffness, $N\text{-}m^2$.

The reference values and constants used for Equation (5-5) are

$\beta = .125\ M^{-2}$ per ^oC

$R_o = 880$ m. (2624 ft = 2.2^o curve)

$U_o = 2 \times 10^7\ N/m^2$ (2900 psi) rated mediocre by SNCF

$\varepsilon = 0.225\ N^{-1/8}\ m^{-1/4}$.

Contrary to what might be expected, the effect of track modulus and rail size on lateral critical force as given in Equation (5-5) is quite small. These conclusions have been verified by measurements [5-8]. An increase in track modulus, U, by a factor of 2 increases the lateral strength by only 9 percent. Increasing the rail size can even reduce the lateral strength for external loads when thermal loads are significant because of the increased thermal forces from larger cross section area of the rail relative to the small increase in lateral stiffness. Consequently, the effects of rail size and track modulus on lateral strength can usually be neglected.

Figure 5-3 shows the effect of rail temperature rise (above the laying temperature) and curve radius for a typical 119-lb rail and a maximum expected rail temperature rise of $\Delta T = 70$ F. These temperature effects are for continuous welded rail (CWR) where thermal stresses are not relieved by rail joints. The calculations have been carried out for a maximum 10-degree curve, (SNCF limits for CWR on curves with a radius smaller than 800 m for wood tie track and 500 m for concrete tie track are shown for reference). These results demonstrate the origin of the 85 percent factor used by the SNCF (see Item 8, Table 5-2) for their design limits in place of the more detailed relation given by Equation (5-5).

To demonstrate further the effect of track construction, curve radius, and axle load on the track lateral force limits, the equations (Items (5) and (7) in Table 5-2)

$$H_c = \frac{P}{3} + 1(10^4) \text{ Newtons for wood ties, and} \qquad (5\text{-}6)$$

$$H_c = \frac{P}{3} + 1.96\ (10^4) \text{ Newtons for concrete ties,} \qquad (5\text{-}7)$$

are recommended for use as track limits for wood and concrete ties at the same tie spacing. These limits were selected as a conservative representation for recently maintained track. The 2:1 unloaded (P = 0) track lateral resistance ratio of wood ties to concrete ties at equal tie spacings agrees rather well with the measured lateral tie resistance data reported previously. However,

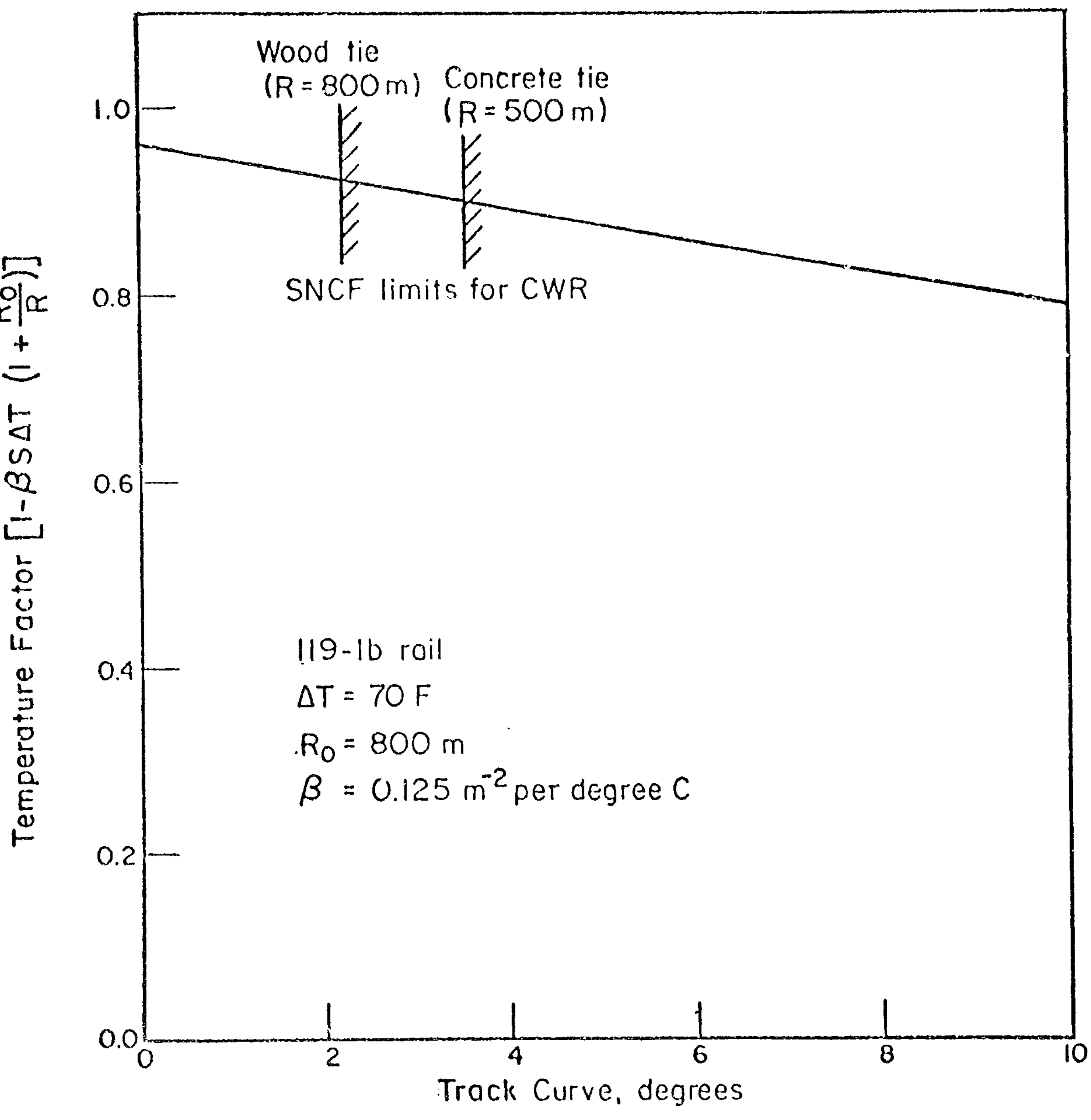


FIGURE 5-3. EFFECT OF CURVE RADIUS ON TRACK LATERAL STRENGTH FOR ΔT=70F RAIL TEMPERATURE (CWR)

the common practice of using wider spacings for concrete ties to compensate for their greater base area reduces the advantage of their lateral strength.

Figure 5-4 shows the track force limits recommended for wood and concrete ties based on Equations (5-6) and (5-7) and the temperature-curvature effect shown in Figure 5-3 for $\Delta T = 70$ F. These same relations are shown in Figure 5-5 to demonstrate that the frequently used guide of $H/P < 0.4$ as safety limit for track lateral shifting is reasonable for heavy axle loads ranging from 40,000 to 60,000 pounds. However, it is somewhat conservative for empty cars and for the lighter axle loads expected for high-speed passenger trains.

5.3 RAIL ROLLOVER

The ratio of lateral L to vertical V wheel loads, L/V, is the principal factor in determining rail lateral deflections. Such deflections can cause derailments by either increasing gage sufficiently to allow the inside wheel to drop off the rail or by displacing the outside rail to a point of total collapse. The resistance to lateral forces depends on a complex combination of lateral bending and torsion of the rail combined with the restoring moment from the vertical forces of adjacent wheels. If a simplified model consisting of one truck on a section of rail with loose joint bars is assumed so that the rail torsional restraint can be neglected, the overturning stability depends only on the rail geometry (ratio of base to height). For conventional rail, this criterion indicates that a ratio of the lateral to vertical wheel forces on one side of a truck in excess of about 0.5 can lead to rail rollover. Of course, this margin is increased considerably by the additional restraint from sound spikes or a good fastener.

5.4 WHEEL DERAILMENT

If the track has sufficient strength, the limiting conditions for train derailment is wheel derailment caused by the wheel climbing the rail. Consequently, the upper bound for track lateral strength requirements is the loading condition resulting in the wheel flange climbing the rail, since any capability in excess of this would be unnecessary.

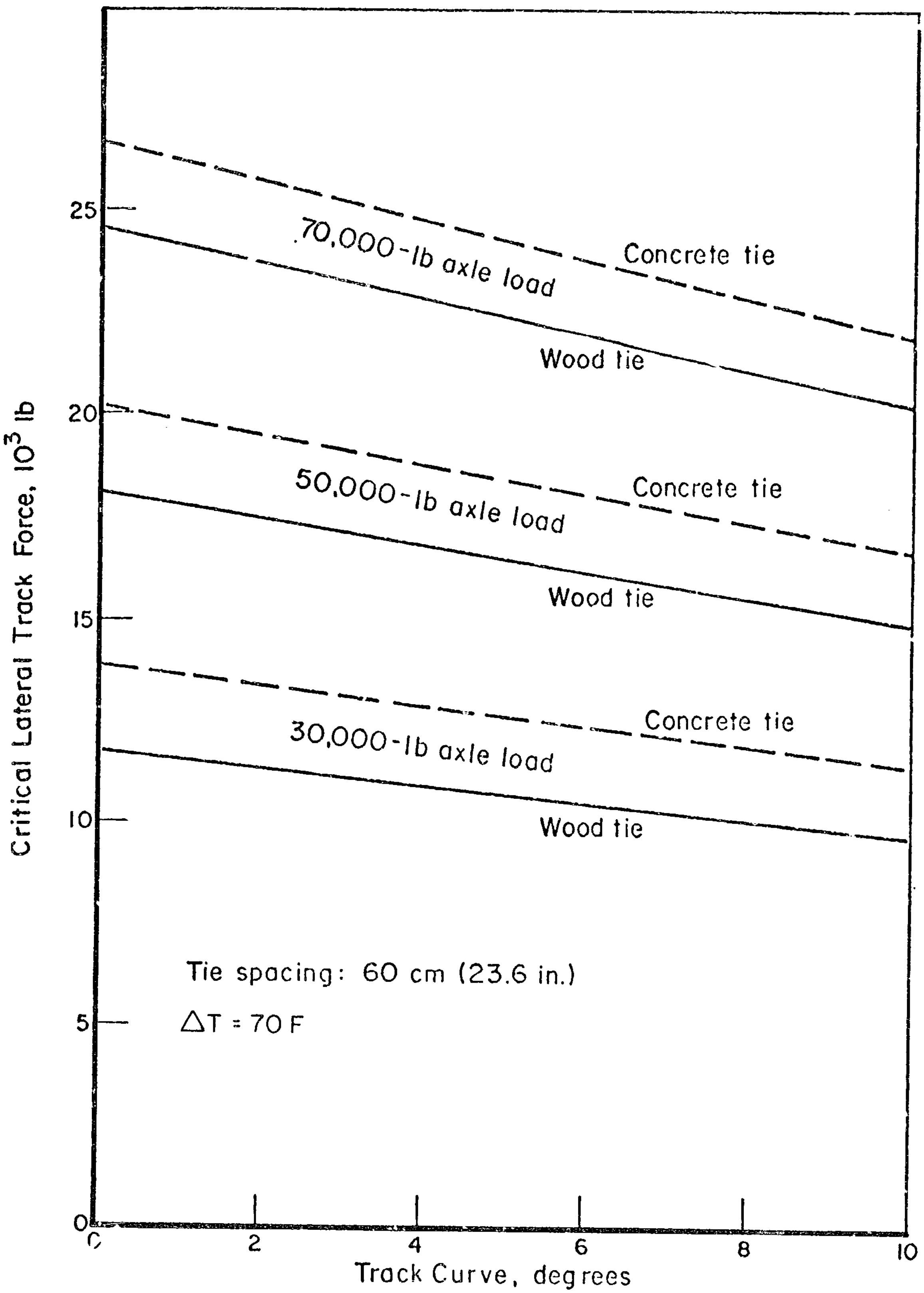


FIGURE 5-4. LATERAL TRACK FORCE LIMITS FOR CURVED TRACK WITH WOOD AND CONCRETE TIES (CWR)

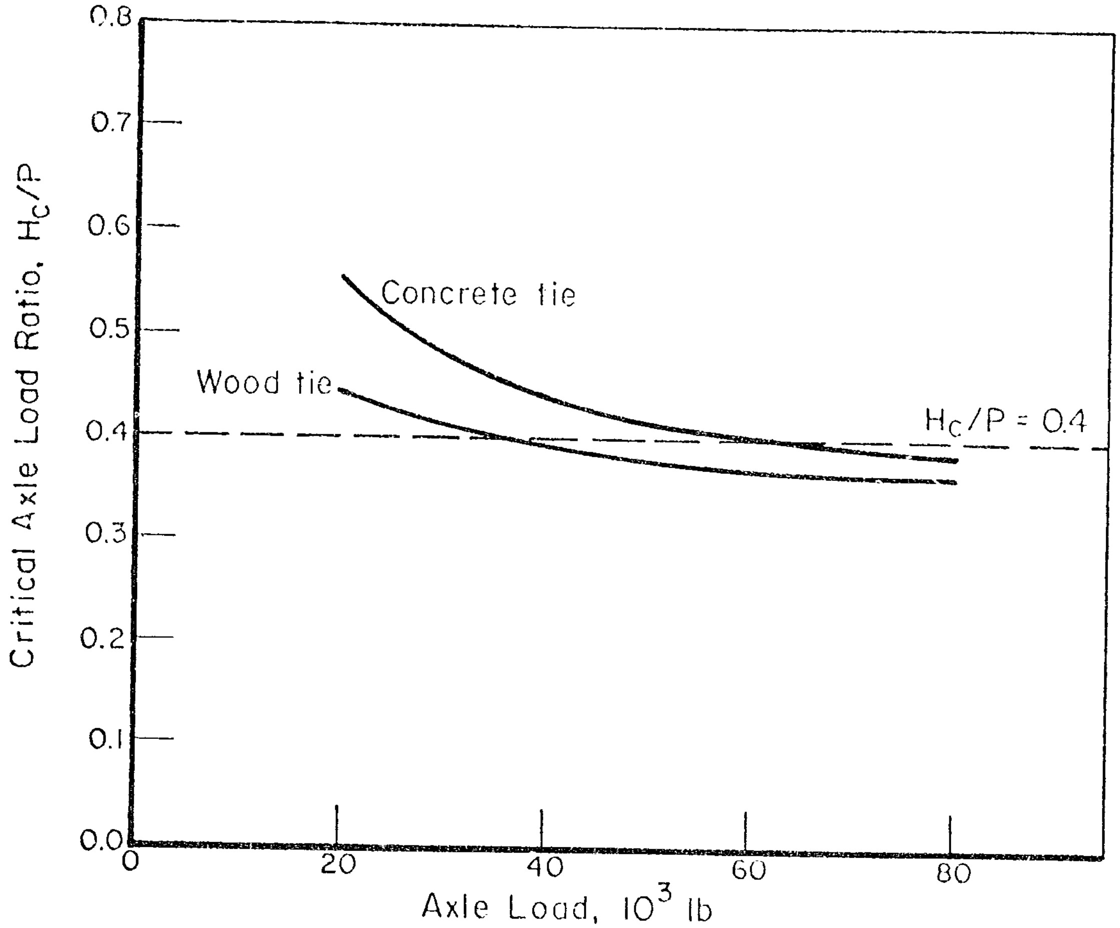


FIGURE 5-5. RATIO OF CRITICAL LATERAL TO VERTICAL AXLE LOADS FOR WOOD AND CONCRETE TIE TRACK

The condition for wheel climb is a complex function of the wheel angle of attack, the local wheel-rail geometry under flange contact, surface conditions which govern friction forces, and the loading dynamics. However, a derailment quotient (ratio of lateral to vertical wheel force L/V) of 0.8 is used as a minimum safety limit by the Japanese National Highway. Derailment quotients as high as 1.0 are often considered acceptable by others, particularly if the loading is more of an impact rather than steady-state loading. Since it takes a finite amount of time for a wheel to climb the rail, steady-state lateral loads are more dangerous than short-duration impacts.

The AREA [5-11] used a capability of maintaining a lateral displacement (gage widening) of no more than 1/4 in. under simultaneous equal lateral and vertical loads (L/V = 1.0) up to the maximum static vehicle wheel load as the basis for determining lateral rail fastener requirements for concrete ties. Therefore, fasteners satisfying this requirement could be expected to maintain gage and resist rail rollover when subjected to lateral wheel loads as high as those which might cause wheel derailment.

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APPENDIX

REPORT OF INVENTIONS

This report contains the formulation of analytical relationships for track performance indices and numerical data from a track response computer program that can be used for design trade-off studies. After a diligent review of the work performed under this contract, it is concluded that no inventions, discoveries, or improvements of inventions were made, however, the results from the parametric study provide a broader and more useful input to track design trade-off and feasibility studies.

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