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TRACK SUPPORT SYSTEMS PARAMETER STUDY

BALLAST AND FOUNDATION MATERIALS
RESEARCH PROGRAM

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16. Abstract

A finite element structural analysis model for conventional railway track support systems (CRTSS) has previously been developed. This study includes parameter studies and sensitivity analyses conducted using the structural model to establish the effects of various parameters on the "instantaneous-elastic" response of CRTSS. The parameters studied are ballast (type and depth), subballast (type and depth), subgrade support conditions, rail size, ties (spacing and width), wheel loading, missing ties and tie type. The parameter study indicates that ballast type and rail size do not significantly effect "instantaneous-elastic" response of CRTSS, while subballast (stabilized), subgrade support condition and wheel loading are some of the major parameters that affect the "instantaneous-elastic" response of the CRTSS.

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PREFACE

This report has been genrated as part of a sub-contract between the Association of American Railroads Research and Test Department, and the University of Illinois.

This sub-contract is part of a larger contract which is a cooperative effort between the Federal Railroad Administration and the Association of American Railroads on improved track structures. The entire program is in response to recognition of the desire for a more durable track structure. To this end, the program is a multi-task effort involving 1) the development of empirical and analytical tools for the description of the track structure so that the economic trade-offs among track construction parameters such as tie size, rail size, ballast depth and cross section, type, subgrade type, stiffness, may be determined. 2) methodologies to upgrade the existing track structures to withstand new demands in loading, 3) development of performance specifications for track components, and 4) investigating the effects of various levels of maintenance.

This particular report describes an analytical tool for investigating track behavior.

A special note of thanks is given to Mr. William S. Autrey, Chief Engineer of Santa Fe, Mr. R. M. Brown, Chief Engineer of Union Pacific, Mr. F. L. Peckover, Engineer of Geotechnical Services, Canadian National Railway, Mr. C. E. Webb, Asst. Vice President, Southern Railway System, as they have served in the capacity of members of the Technical Review Committee for this Ballast and Foundation Materials Program, and Dr. R. M. McCafferty as the Contracting Officer's Technical Representative of the FRA on the entire research program.

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Director-Dynamics Research
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Track Structures Research Program
Association of American Railroads

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LIST OF ABBREVIATIONS AND SYMBOLS

E = Young's modulus

E_R = resilient modulus

I = moment of inertia of rail section

μ = Poisson's ratio

σ_1 = major principal stress

σ_2 = intermediate principal stress

σ_3 = minor principal stress

$\sigma_D = \sigma_1 - \sigma_3$ = deviator stress

$\theta = \sigma_1 + \sigma_2 + \sigma_3$ = sum of principal stresses

$= \sigma_1 + 2\sigma_3$ in a triaxial test

CHAPTER 1

INTRODUCTION

1.1 General

The Department of Civil Engineering of the University of Illinois at Urbana-Champaign is currently conducting a broad based research program in the areas of ballast and foundation (subgrade) materials. The University of Illinois is serving as a sub-contractor to the Association of American Railroads. The Ballast and Foundation Materials Research Program is sponsored by the U. S. Department of Transportation, Federal Railroad Administration.

A structural analysis model (ILLI-TRACK) for conventional railway track support systems has previously been developed in Phase II of the research program (1). Phase III of the research program includes parameter studies and sensitivity analyses to establish the effects of various parameters on the response of conventional railway track support systems.

Those parameters to be considered in Phase III are:

1. Ballast (type and depth)
2. Subballast (type and depth)
3. Subgrade support
4. Rail size
5. Ties (spacing and width)
6. Wheel loading
7. Missing ties
8. Type of ties

It is anticipated that Phase III results will be of great value relative to:

1. Establishing the degree of accuracy required for adequately characterizing the various material properties.
2. Considering the relative effects of material property changes (ballast, subballast, subgrade) that may occur during the track service life.
3. Developing improved track structure response through the judicious selection of ballast and subballast materials and rail-tie systems.
4. Identifying procedures and practices that may beneficially affect track structure response.
5. Establishing materials criteria, design guidelines, construction requirements, etc.

1.2 Report Organization

A brief review of the ILLI-TRACK analysis model is presented in Chapter 2. Details of the parameter study, results of the various analyses, and a discussion of the results are included in Chapter 3. Chapter 4 presents a summary of findings and conclusions derived from the parameter study.

CHAPTER 2

STRUCTURAL ANALYSIS MODEL FOR CONVENTIONAL RAILWAY TRACK SUPPORT SYSTEMS

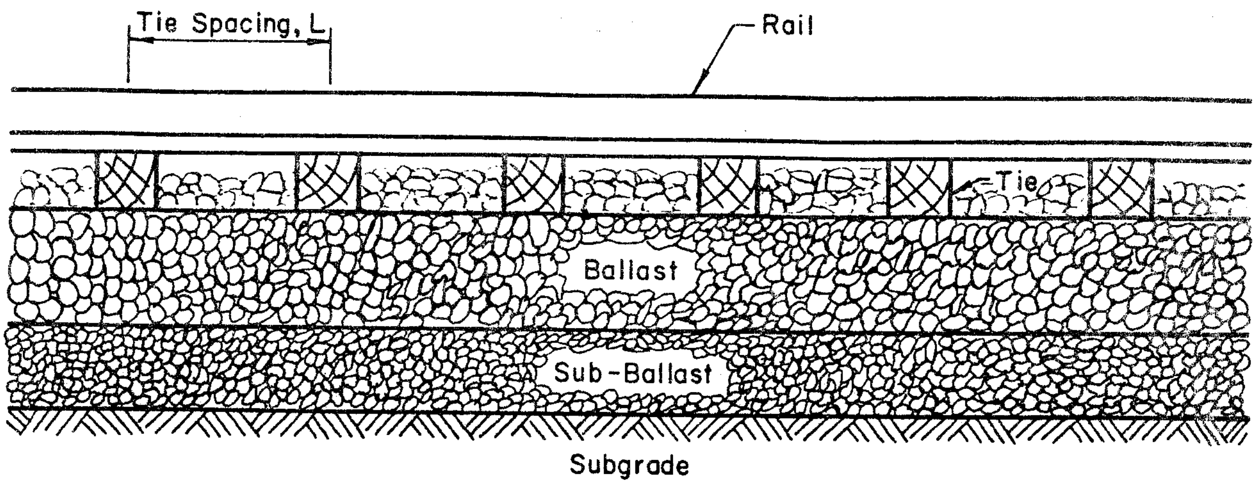
2.1 General

One of the integral parts of the "Ballast and Foundation Materials Research Program" is the development of a "mechanistic structural analysis model" of conventional railway track support systems (CRTSS). The analysis model has been developed using the "finite element" method. The model provides for adequate engineering analysis of the track system response due to applied vertical wheel loading. The model incorporates the basic components of the CRTSS and can accommodate "stress-dependent" structural response characteristics of the ballast, the subballast, and the subgrade. A detailed discussion of the development of the finite element model is given in Reference 1. A brief description of the model is presented below.

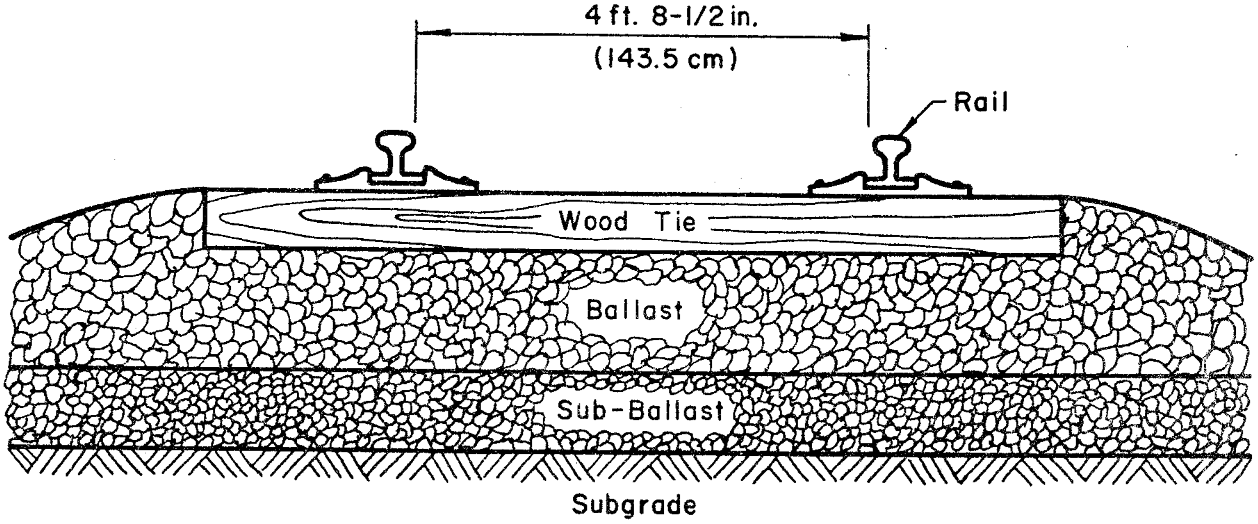
2.2 Brief Description of the Model

A typical longitudinal section and a typical transverse section of a contentional railway track are shown in Figure 2.1. It can be seen that because of the three-dimensional geometry and non-uniform loading conditions, analysis of the conventional railway track structure should consider a three-dimensional approach. While it is possible to formulate a three-dimensional finite element model that would represent the system, the amount of discretization and the computer costs required for solution of the problem would be high and probably impractical.

If the symmetrical nature of the loading in the transverse direction is examined, Figure 2.1, it is apparent that a two stage analysis might provide a reasonable engineering approach. In this two stage



(a) Longitudinal Section



(b) Transverse Section

Figure 2.1 A Typical Longitudinal and a Typical Transverse Section of a Conventional Railway Track.

analysis, a longitudinal analysis is performed followed by a transverse analysis.

The longitudinal analysis considers point loads (corresponding to wheel loads), acting on a single rail sitting on the tie-ballast-subgrade system. Figure 2.2 shows a typical finite element mesh used for the longitudinal analysis. The rail-tie subsystem is represented as a continuous beam supported on tie springs. Rectangular planar elements are used to represent the ballast, the subballast, and the subgrade. The thickness of the elements is varied with depth using a "pseudo" plane strain technique (Discussed in Ref. 1) to account for the spread of loading in the direction perpendicular to the plane. This allows a three-dimensional load spread, which is known to exist in practice, to be simulated with a two-dimensional model. The displacement components are assumed to vary linearly over each element.

The transverse analysis uses the output from the longitudinal analysis for input. Either the maximum reaction or the maximum deflection at a tie obtained from the longitudinal analysis is used as input at a tie which rests on the ballast-subgrade system. Again the "pseudo" plane strain technique is used. The tie can be represented either as a two-dimensional body or as a beam resting on the ballast. Rectangular element representation is used for the ballast, the subballast and the subgrade materials, and the displacement components are assumed to vary linearly over each element. Figure 2.3 shows the finite element mesh used for the transverse analysis. Triangular elements can be used to incorporate sloping ballast shoulders.

The finite element model has been validated using measured response at Section 9 of the Kansas Test Track (Ref. 1). Good agreement was obtained between the measured response and that calculated using the

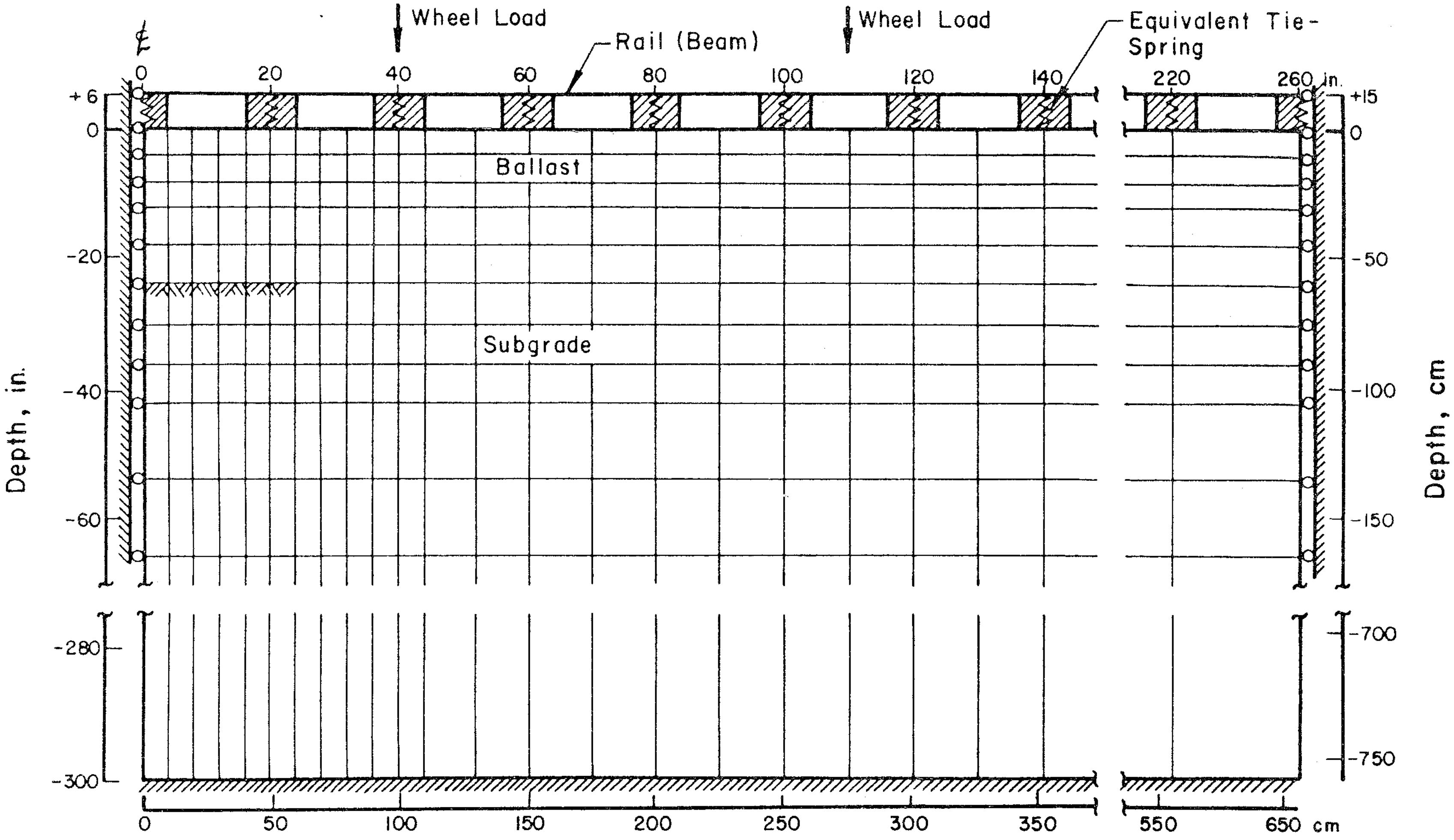
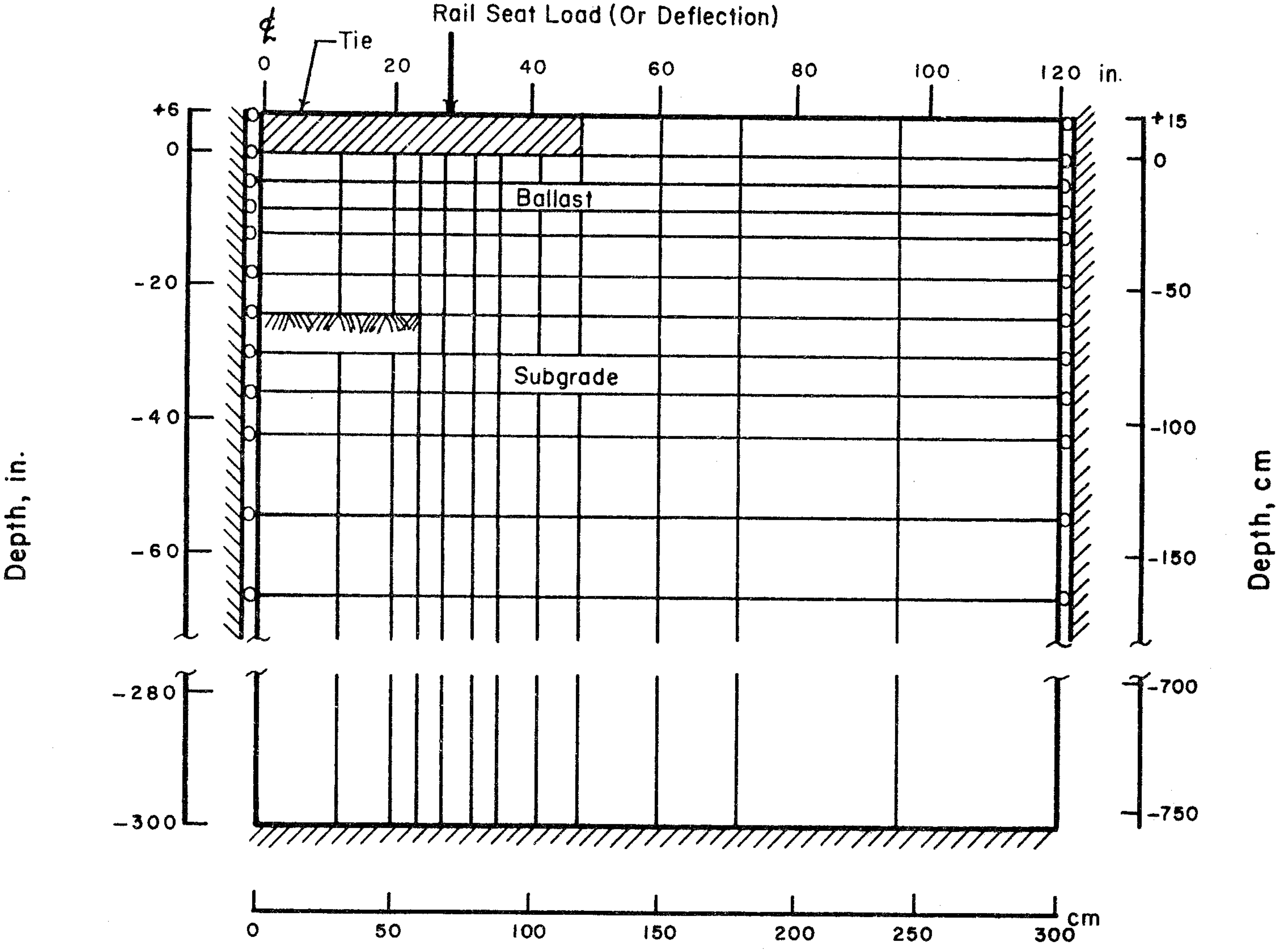


Figure 2.2 A Typical Finite Element Mesh Used for Longitudinal Analysis.



Note: Triangular elements can be used to incorporate sloping ballast shoulder.

Figure 2.3 A Typical Finite Element Mesh Used for Transverse Analysis finite element model. As more appropriate field response data become available, it is expected that the model can be further validated. In its present stage, the ILLI-TRACK model is not a design model but rather an analysis tool and this should be borne in mind when using or interpreting the results as given by ILLI-TRACK.

CHAPTER 3

PARAMETER STUDY

3.1 General

An important feature of the design of the track support system is the judicious selection of the optimum design based on factors such as available resources, anticipated performance, and level of service requirements. One method used in aiding the selection of an optimum design is to conduct a parameter study or a sensitivity analysis and to evaluate the effects of critical design parameters on the response of the track support system. The structural model used to evaluate the response of the track support system should therefore be capable of incorporating the critical design parameters. The finite element model described in Chapter 2 possesses this capability and was used to study the effects of various design parameters on the response of the track support system.

Initially the finite element model was checked for its sensitivity to certain input parameters such as initially assumed moduli values for ballast and subgrade and failure criteria in terms of the allowable stress ratio, σ_1/σ_3, and the allowable minimum principal stress, σ_3, for the ballast.

3.2 The Reference Track

For the purpose of the parameter study, a reference track was designated to allow the comparison of the structural response of track support systems with different design parameters. Characteristics of the reference track are given below.

<u>Rail</u> - 136 lb/yd (68 kg/m)

$I = 94.90 \text{ in}^4$ (3954 cm^4)

$E = 30{,}000$ ksi ($207{,}000 \text{ MN/m}^2$)

Ties - Timber ties

Width = 8 in. (20.3 cm)

Thickness = 7 in. (17.8 cm)

Length = 8 ft (2.44 m)

Tie Spacing = 20 in. (50.8 cm)

Compressive Modulus = 1,250 ksi (8618 MN/m^2)

Effective bearing length under each rail = 18 in. (45.7 cm)

Ballast - Crushed stone ballast, AREA #4 Gradation

Resilient Response Model: $E_R = 5082\ (\theta)^{0.58}$

$\mu = 0.35$

Ballast Depth = 12 in. (30.5 cm)

Subballast - none

Subgrade (Embankment Material)

$\mu = 0.47$

Resilient Response Curve Data: (Average Subgrade)

σ_D, psi (kN/m^2)	E_R, psi (kN/m^2)
0.1 (0.7)	14820.0 (102180.0)
6.2 (42.8)	8000.0 (55160.0)
36.2 (249.6)	2900.0 (19990.0)

The reference loading was taken to be two trucks of two adjacent freight cars, each car having a gross weight of 240,000 lb. (108800 kg), thus giving approximate wheel load of 30,000 lb (13600 kg). The truck spacing equalled 150 in. (3.81 m) and the axle spacing equalled 70 in. (1.78 m).

The structural responses of particular interest were considered to be:

1. Rail deflection
2. Rail moment
3. Ballast vertical stress
4. Subgrade vertical stress
5. Subgrade vertical strain
6. Failure pattern in ballast (if significant)

The response of the reference track support system to the reference loading is detailed in Table 3.1 for the following initially assumed moduli values for ballast and subgrade.

Section 1: Initially assumed ballast modulus = 30 ksi (207 MN/m^2)
Initially assumed subgrade modulus = 5 ksi (34.5 MN/m^2)
Section 2: Initially assumed ballast modulus = 30 ksi (207 MN/m^2)
Initially assumed subgrade modulus = 10 ksi (69 MN/m^2)
Section 3: Initially assumed ballast modulus = 20 ksi (138 MN/m^2)
Initially assumed subgrade modulus = 5 ksi (34.5 MN/m^2)

It can be seen from Table 3.1 that the initially assigned moduli values for ballast and subgrade, when properly selected, do not exert significant influence on the final response of the track support system.

The effect of varying the failure criteria for ballast in the reference track support system subjected to the reference loading is detailed in Table 3.2 for the following five conditions:

Section 1: Maximum allowable stress ratio, σ_1/σ_3 = 10
Minimum allowable principal stress, σ_3 = 0 psi (0 kN/m^2)
Section 2: Maximum allowable stress ratio, σ_1/σ_3 = 5
Minimum allowable principal stress, σ_3 = 0 psi (0 kN/m^2)
Section 3: Maximum allowable stress ratio, σ_1/σ_3 = 15
Minimum allowable principal stress, σ_3 = 0 psi (0 kN/m^2)

Table 3.1 Comparison of Sections with Different Initially Assumed Moduli Values for Ballast and Subgrade

	Initially Assumed Moduli Values		
	Section 1	Section 2	Section 3
Maximum Rail Deflection, in.	0.10	0.10	0.09
(mm)	2.5	2.5	2.3
Maximum Rail Moment, lbf in. (x1000)	297	276	277
(kNm)	33.6	31.2	31.3
Maximum Ballast Vertical Stress, psi	114.3	111.2	105.0
(kN/m^2)	788.1	766.7	721.0
Maximum Subgrade Vertical Stress, psi	26.4	24.8	23.5
(kN/m^2)	182.0	171.0	162.0
Maximum Subgrade Vertical Strain ($x10^{-4}$)	11.5	11.3	10.3

Table 3.2 Comparison of Sections with Different Failure Criteria for Ballast

	Sec. 1	Sec. 2	Sec. 3	Sec. 4	Sec. 5
Maximum Rail Deflection, in.	0.10	0.10	0.09	0.10	0.07
(mm)	2.5	2.5	2.3	2.5	1.8
Maximum Rail Moment, lbf in. (x1000)	297	262	279	297	245
(kNm)	33.6	29.6	31.5	33.6	27.7
Maximum Ballast Vertical Stress, psi	114.3	70.7	68.1	114.2	54.5
(kN/m^2)	788.1	487.5	469.5	787.4	375.8
Maximum Subgrade Vertical Stress, psi	26.4	20.9	22.4	26.4	19.6
(kN/m^2)	182.0	144.1	154.1	182.0	135.1
Maximum Subgrade Vertical Strain ($x 10^{-4}$)	11.5	9.8	7.5	11.5	7.5

Section 4: Maximum allowable stress ratio, σ_1/σ_3 = 10

Minimum allowable princiaal stress, σ_3 = -2 psi (-13.8 kN/m^2)

Section 5: Maximum allowable stress ratio, σ_1/σ_3 = 999999

Minimum allowable principal stress, σ_3 = -5.0 psi (-34.5 kN/m^2)

It should be noted that when any of the failure criteria is exceeded by a ballast element, the ballast element is assigned a modulus of a lower value to be used in the next incremental step analysis. The stress ratio criteria relates to the shear strength of the ballast material and the minimum principal stress criteria relates to the inability of open graded ballast to resist tensile stress. A very high stress ratio and a negative allowable minimum principal stress would be characteristic of well bound (or cementitious) materials.

Variation in failure criteria significantly affected the response of the track support system. For the purpose of the parameter study it was decided to use initially assumed moduli values for ballast and subgrade equal to 30 ksi (207 MN/m^2) and 5 ksi (34.5 MN/m^2), respectively. The failure criteria selected for the ballast were maximum allowable stress ratio, σ_1/σ_3, of 10 and a minimum allowable principal stress, σ_3 of 0 psi (0 kN/m^2). The results of the parameter study are discussed in the subsequent sections. When considering the effect of any design parameter, only the input value of the design parameter was changed and all other design factors were kept constant.

3.3 Ballast as Variable

3.3.1 Ballast Depth

The following depths of crushed stone ballast were considered:

Section 1: 8 in. (20.3 cm)

Section 2: 12 in. (30.5 cm) - reference section

Section 3: 24 in. (61.0 cm)

The effect of depth of ballast was evaluated for two types of rails - 136 lb/yd (68 kg/m) rail and 115 lb/yd (57 kg/m) rail. The response of the track system is shown in Figure 3.1 for the 136 lb/yd (68 kg/m) rail. There was little difference in response for the 8 in. (20.3 cm) and the 12 in. (30.5 cm) ballast, but a reduction in rail deflection and the subgrade stress was obtained by using a 24 in. (61.0 cm) ballast. Pertinent results are summarized in Table 3.3.

The effects of changing ballast depth when 115 lb/yd (57 kg/m) rail was used are shown in Figure 3.2. It can be seen that for the less "stiff" rail, the ballast depth does exert considerable influence on rail deflection and subgrade stress. Rail moment tends to remain relatively constant for the different depths of ballast. Pertinent results are summarized in Table 3.4.

The 24 in. (61.0 cm) thick ballast layer tends to transmit the vertical stress on the subgrade more uniformly than the thinner ballast layers for both the rail types considered.

To evaluate the effect of using a stabilized soil layer, similar analyses were conducted for the 136 lb/yd (68 kg/m) rail and the three ballast depths but a 6 in. (15.2 cm) layer of stabilized soil subballast with a constant modulus value of 50 ksi (345 MN/m^2) was incorporated into the sections. The results are shown in Figure 3.3.

The use of the stabilized soil layer tends to minimize the differences in the structural response due to changes in the ballast depth. Pertinent results are summarized in Table 3.5.

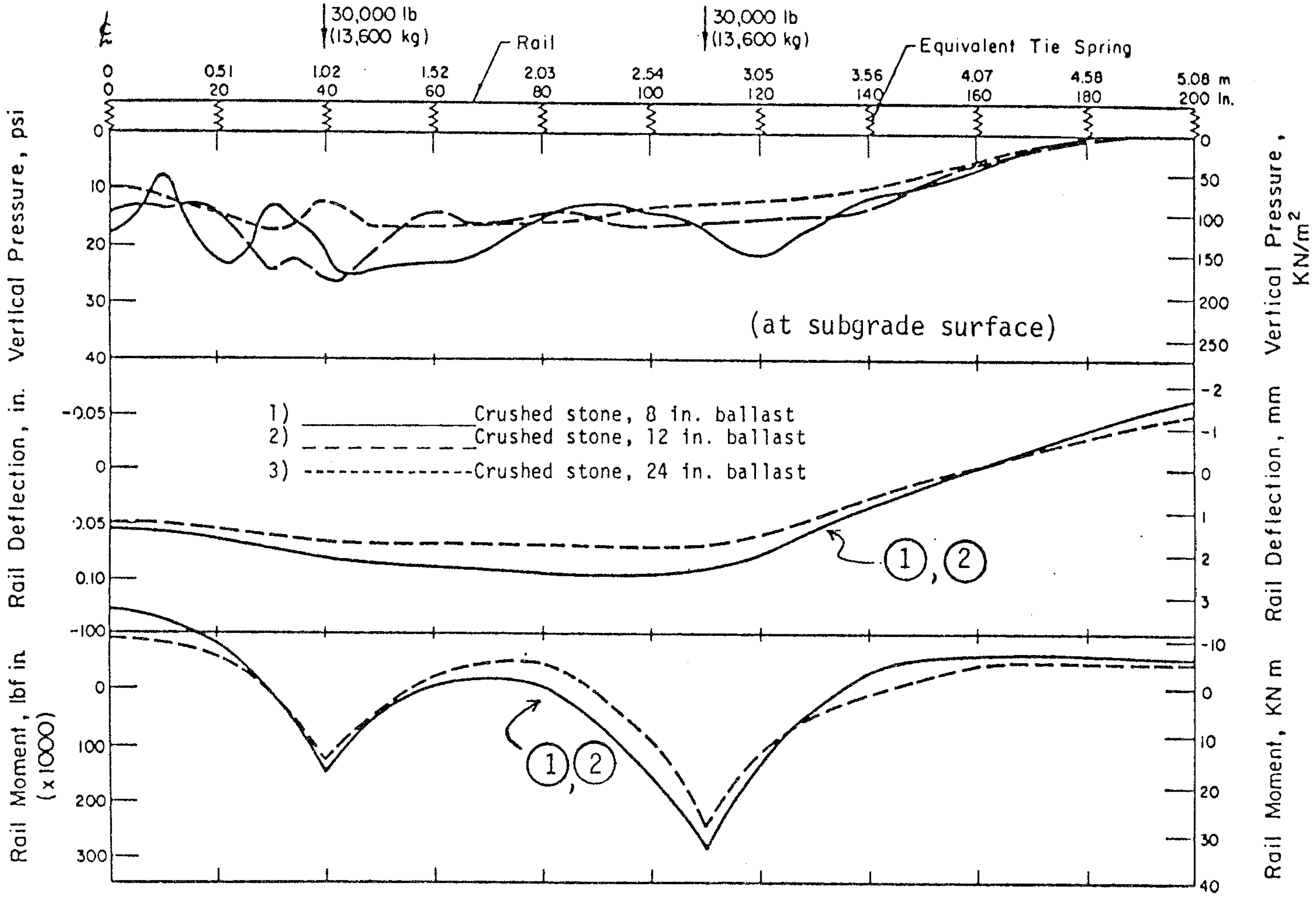


Figure 3.1 Comparison of Sections with Ballast Depth as the Variable (136 lb/yd rail)

Table 3.3 Comparison of Sections with Ballast Depth as the Variable (136 lb/yd rail)

	Ballast Depth (136 lb/yd rail)		
	8 in.	12 in.	24 in.
Maximum Rail Deflection, in.	0.10	0.10	0.07
(mm)	2.5	2.5	1.8
Maximum Rail Moment, lbf in. (x1000)	293	297	259
(kNm)	33.1	33.6	29.3
Maximum Ballast Vertical Stress, psi	88.0	114.3	91.3
(kN/m^2)	606.8	788.1	633.0
Maximum Subgrade Vertical Stress, psi	24.2	26.4	17.6
(kN/m^2)	166.9	182.0	121.4
Maximum Subgrade Vertical Strain ($\times 10^{-4}$)	10.9	11.5	6.5

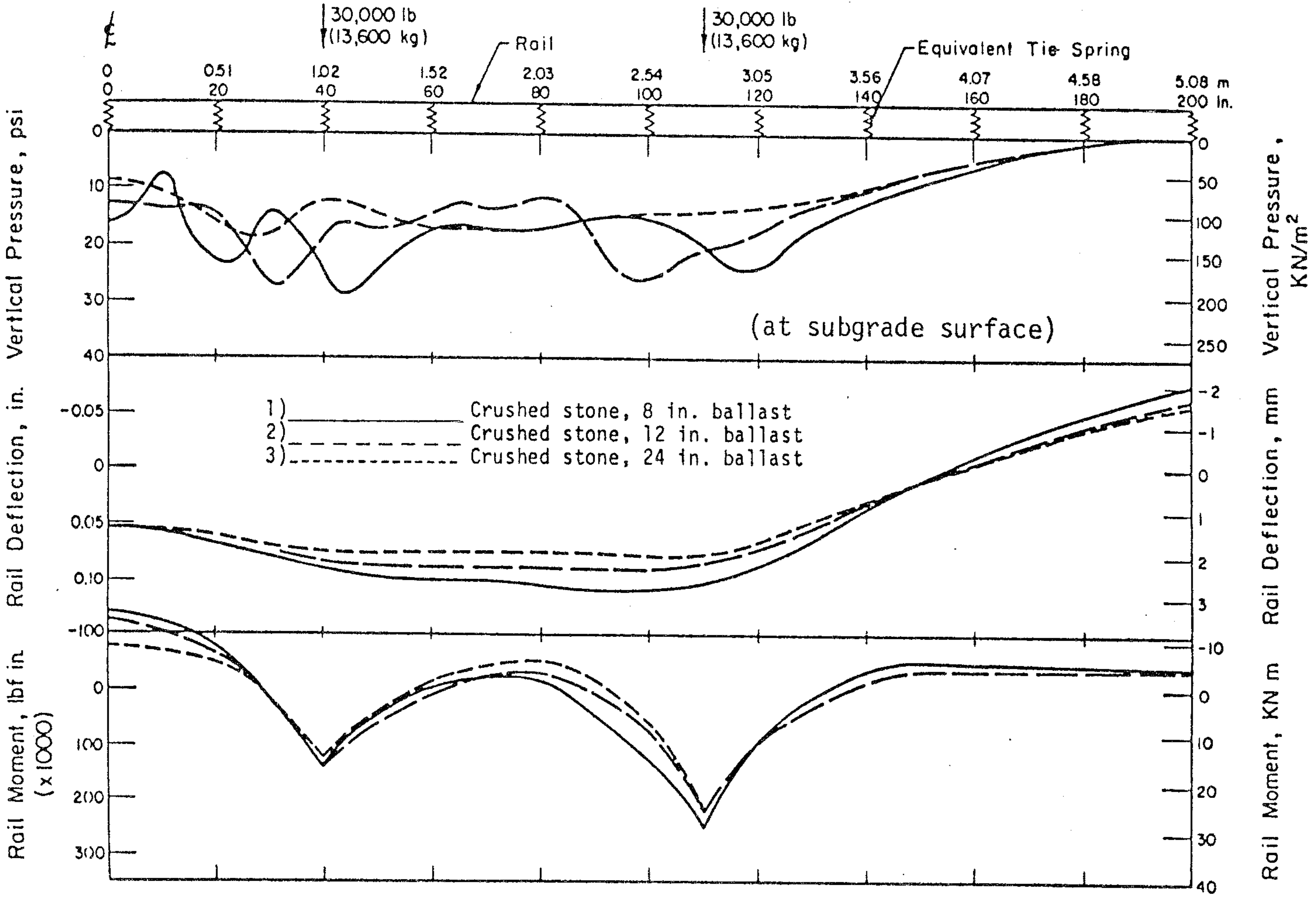


Figure 3.2 Comparison of Sections with Ballast Depth as the Variable (115 lb/yd rail)

Table 3.4 Comparison of Sections with Ballast Depth as the Variable (115 lb/yd rail)

	Ballast Depth (115 lb/yd rail)		
	8 in.	12 in.	24 in.
Maximum Rail Deflection, in.	0.11	0.09	0.08
(mm)	2.8	2.3	2.0
Maximum Rail Moment, lbf in. (x1000)	266	227	235
(kNm)	30.1	25.6	26.6
Maximum Ballast Vertical Stress, psi	108.9	94.2	109.8
(kN/m^2)	750.9	649.5	757.1
Maximum Subgrade Vertical Stress, psi	28.0	26.4	18.9
(kN/m^2)	193.1	182.0	130.3
Maximum Subgrade Vertical Strain ($x10^{-4}$)	15.2	9.4	7.8

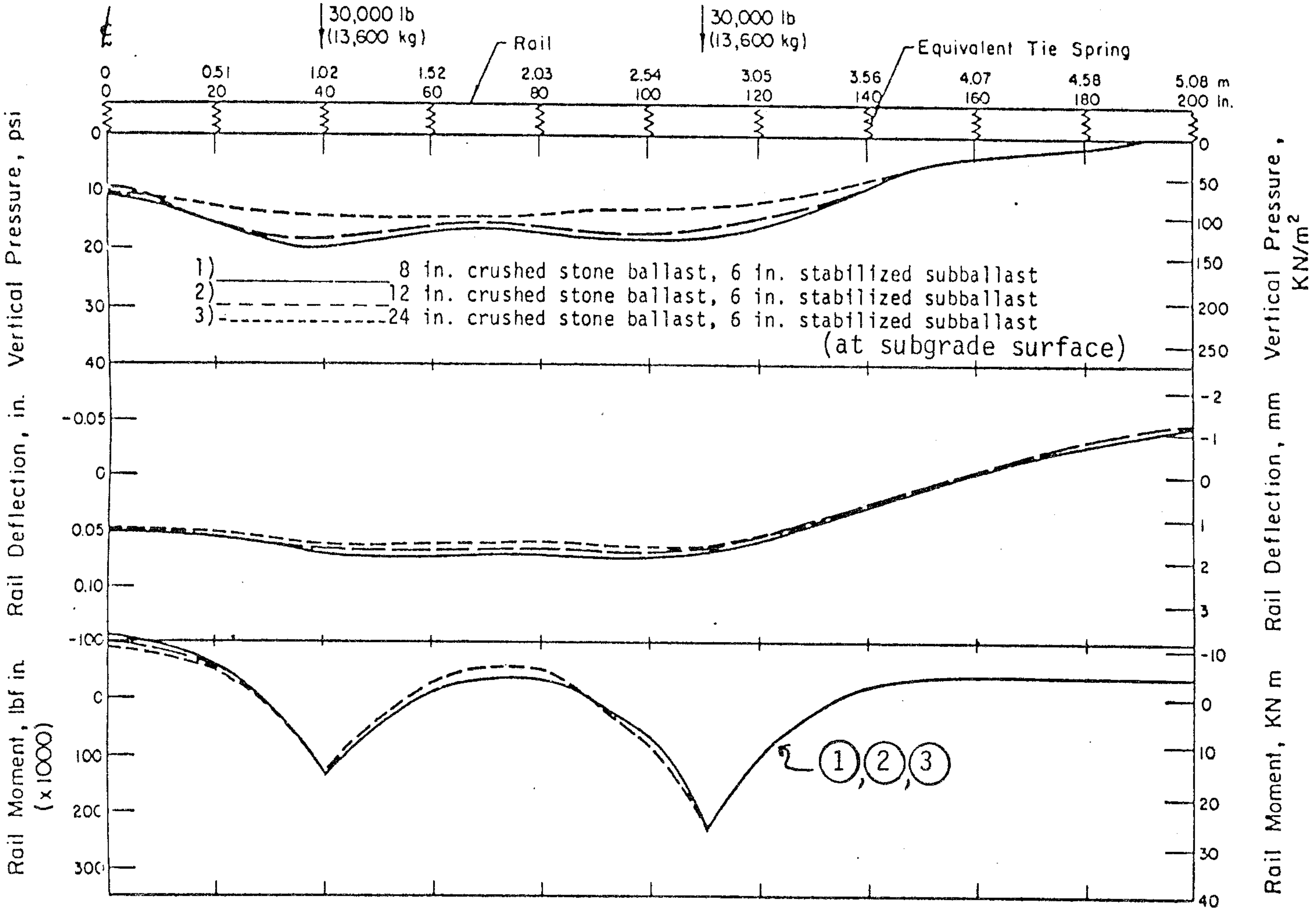


Figure 3.3 Comparison of Sections (incorporating stabilized subballast) with Ballast Depth as the Variable (136 lb/yd rail)

Table 3.5 Comparison of Sections (incorporating stabilized subballast) with Ballast Depth as the Variable (136 lb/yd rail)

	Ballast Depth (136 lb/yd rail) (with 6 in. (15.3 cm) stabilized Layer)		
	8 in.	12 in.	24 in.
Maximum Rail Deflection, in.	0.07	0.07	0.07
(mm)	1.8	1.8	1.8
Maximum Rail Moment, lbf in. (x1000)	250	240	243
(kNm)	28.2	27.1	27.5
Maximum Ballast Vertical Stress, psi	82.2	86.4	92.1
(kN/m^2)	566.8	595.7	635.0
Maximum Subgrade Vertical Stress, psi	20.1	18.8	14.8
(kN/m^2)	138.6	129.6	102.0
Maximum Subgrade Vertical Strain ($\times 10^{-4}$)	7.9	7.1	4.8

3.3.2 Ballast Type

Section 1: Crushed Stone Ballast, AREA #4 Gradation - reference section

Resilient Response Model - $E_R = 5082\ (\theta)^{0.58}$

Section 2: Blast Furnace Slag Ballast, AREA #4 Gradation

Resilient Response Model - $E_R = 1957\ (\theta)^{0.77}$

Section 3: Well Graded Ballast - CA 10 (Maximum Size 1 in. (2.5 cm))

Resilient Response Model - $E_R = 3582\ (\theta)^{0.59}$

The comparison of the responses of the above three sections, as shown in Figure 3.4 indicate that the influence of ballast type on the transient response of the track support system is not great. However, different ballast materials exhibit characteristically different permanent deformation (rutting or loss of alignment) behavior and particle breakdown (degradation) when subjected to repeated application of a particular state of stress. While the transient response using different ballasts may be similar, the ballasts may possess different durability properties. Therefore, it is essential when comparing ballast types to consider the factors affecting long term behavior of the ballast in addition to transient structural response.

Pertinent results for the three ballast types considered are summarized in Table 3.6.

3.4 Subballast as Variable

The materials normally considered for use as subballast are lower quality granular materials (usually used as a filter layer) or stabilized soil. The thicknesses of subballast commonly in use range from about 6 in. (15.3 cm) to about 12 in. (30.5 cm). Several types of subballast and depths of subballast were considered in the parameter study.

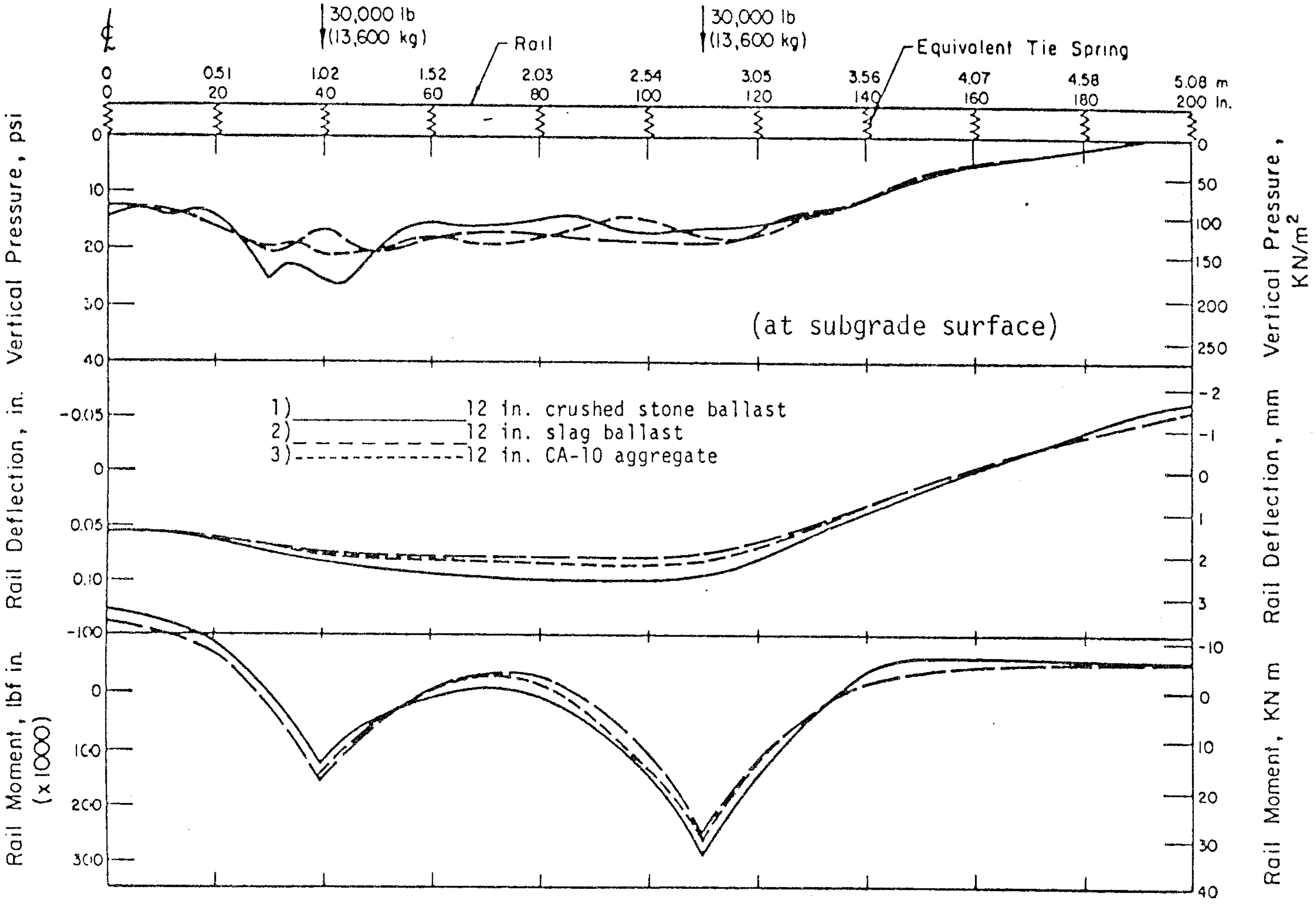


Figure 3.4 Comparison of Sections with Ballast Type as the Variable.

Table 3.6 Comparison of Sections with Ballast Type as the Variable

	Ballast Type		
	C. Stone	Slag	Well Graded
Maximum Rail Deflection, in.	0.10	0.08	0.09
(mm)	2.5	2.0	2.3
Maximum Rail Moment, lbf in. (x1000)	297	260	279
(kNm)	33.6	29.3	31.5
Maximum Ballast Vertical Stress, psi	114.3	80.3	81.8
(kN/m^2)	788.1	553.7	564.0
Maximum Subgrade Vertical Stress, psi	26.4	20.3	20.8
(kN/m^2)	182.0	140.0	143.4
Maximum Subgrade Vertical Strain (x10^{-4})	11.5	7.1	8.3

3.4.1 Subballast Type

Section 1: No subballast - reference section

Section 2: 6 in. (15.2 cm) stabilized soil layer
Constant E = 50 ksi (345 MN/m^2)

Section 3: 6 in. (15.2 cm) stabilized soil layer
Constant E = 1,000 ksi (690 MN/m^2)

Section 4: 6 in. (15.2 cm) sandy subballast
Resilient Response Model - $E_R = 6700\ (\theta)^{0.36}$(Ref. 2)

The effect of subballast type is shown in Figures 3.5 and 3.6. The inclusion of the stabilized soil layer has a large influence on the structural response of the track support system. The rail deflection and rail moments are reduced and the vertical stress is uniformly transmitted to the subgrade. The section without any subballast has localized zones of high vertical stresses at the subgrade surface under the tie below the wheel load. As shown in Figure 3.6 the stabilized layer has a dramatic effect on the occurrence of failure zones in the ballast layer. Failure zones in the ballast layer are reduced using a stabilized soil layer with a constant modulus of 50 ksi (345 MN/m^2) and are almost non-existent when a constant modulus of 1,000 ksi (6900 MN/m^2) is used for the stabilized soil layer. The slab-type behavioral mechanism of the stiffer stabilized layer tends to minimize the development of tensile stresses or dilatational tendency within the ballast layer, thus allowing the ballast material to achieve higher moduli values. There is also a significant reduction in the vertical strain at the surface of the subgrade in the stabilized layer sections. Pertinent results are summarized in Table 3.7.

3.4.2 Stabilized Subballast Depth

The following subballast depths were considered:

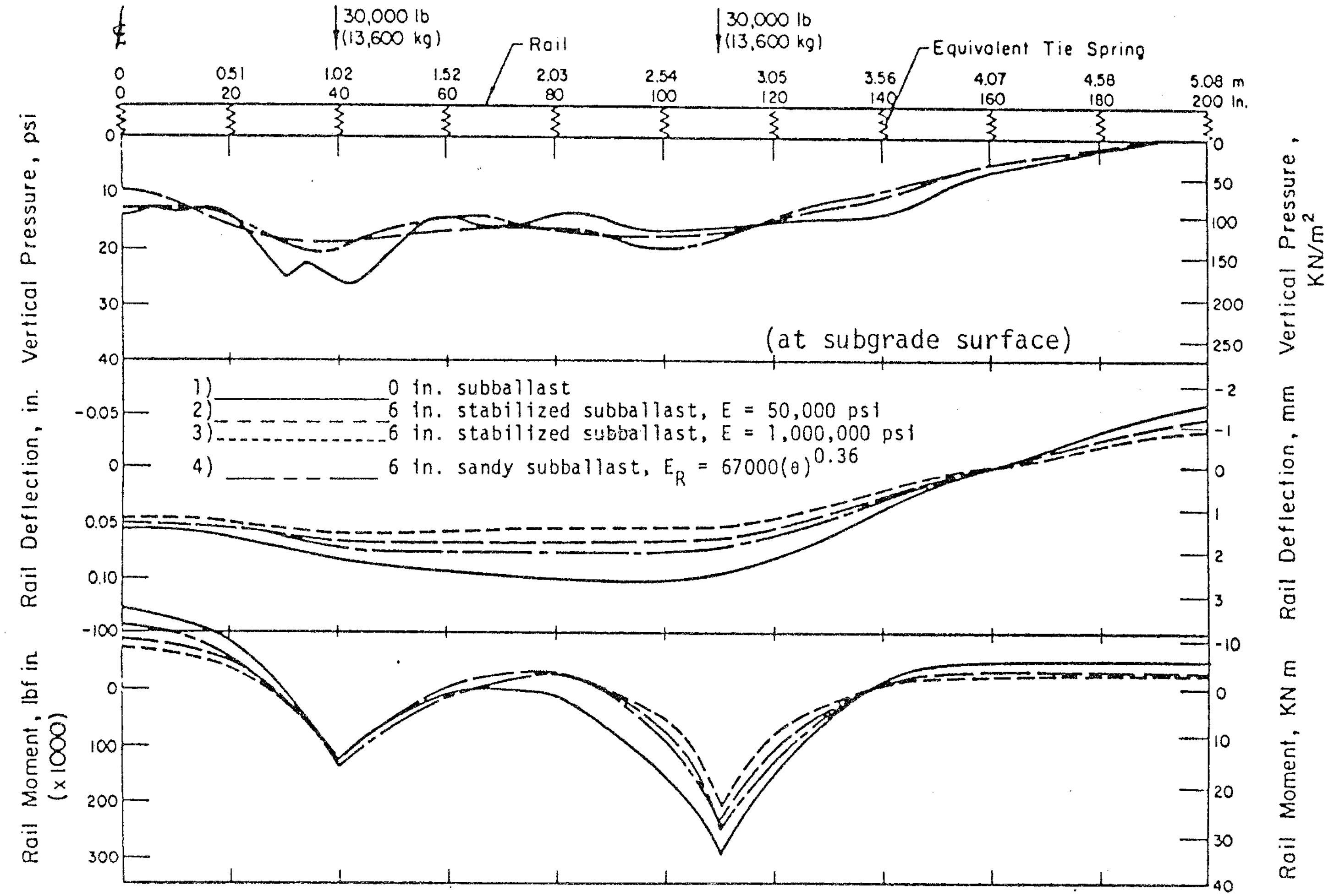


Figure 3.5 Comparison of Sections with Subballast Type as the Variable.

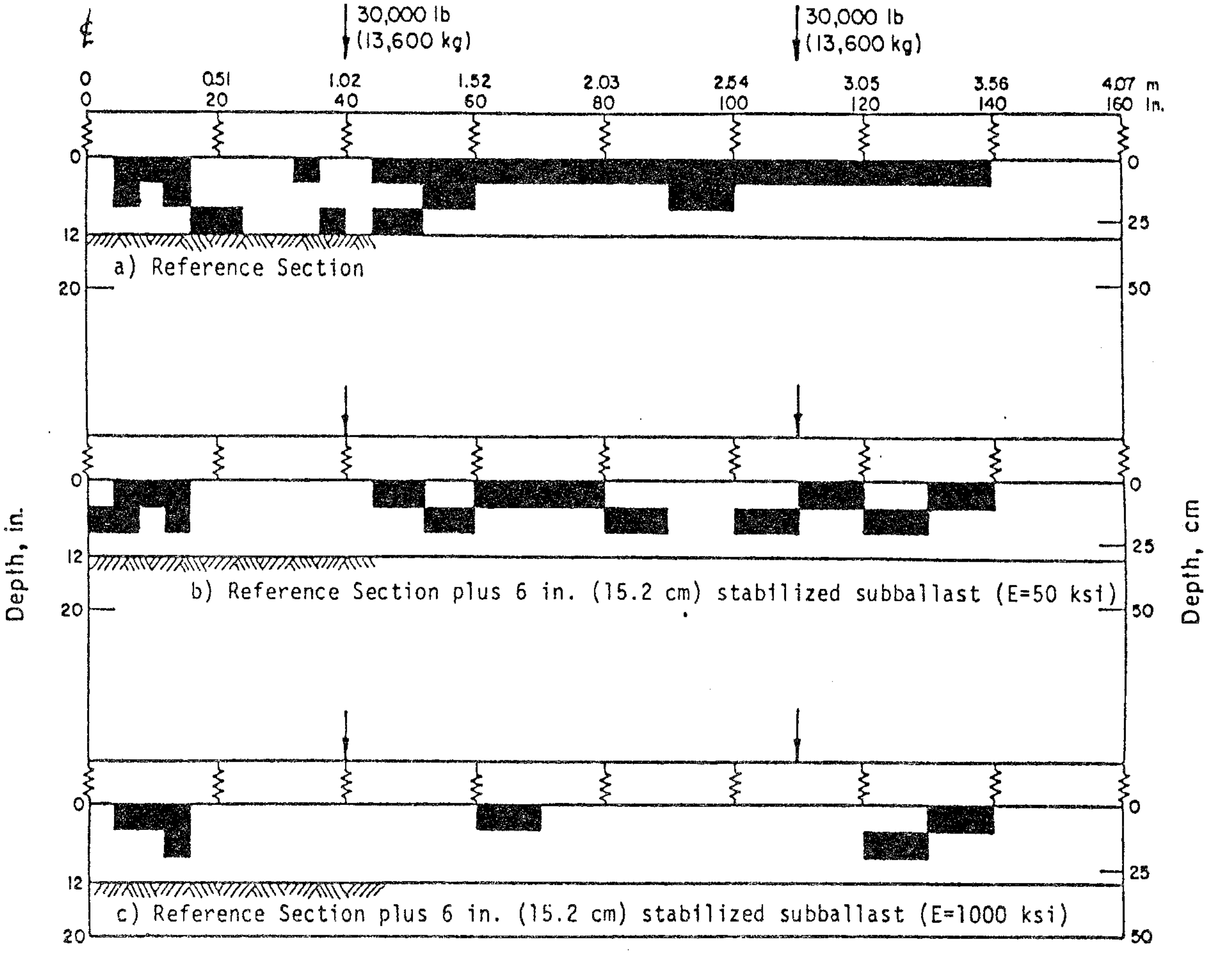


Figure 3.6 Location of Failure Zones in the Ballast Layers

Table 3.7 Comparison of Sections with Subballast Type as the Variable

	Subballast Type (6 in. thick)			
	No Subballast	Stabilized E = 50 ksi	Stabilized E = 1000 ksi	Sandy Subballast
Maximum Rail Deflection, in.	0.10	0.07	0.06	0.08
(mm)	2.5	1.8	1.5	2.0
Maximum Rail Moment, lbf in. (x1000)	297	240	209	251
(kNm)	33.6	27.1	23.6	28.4
Maximum Ballast Vertical Stress, psi	114.3	86.4	78.0	107.1
(kN/m^2)	788.1	595.7	537.8	738.5
Maximum Subgrade Vertical Stress, psi	26.4	18.8	17.8	20.1
(kN/m^2)	182.0	129.6	122.7	138.6
Maximum Subgrade Vertical Strain ($x10^{-4}$)	11.5	7.1	4.2	7.7

Section 1: No subballast - reference section

Section 2: 6 in. (15.2 cm) stabilized soil layer
Constant E = 50 ksi (345 MN/m^2)

Section 3: 12 in. (25.4 cm) stabilized soil layer
Constant E = 50 ksi (345 MN/m^2)

The effect of the stabilized subballast depth is shown in Figure 3.7. As mentioned in the previous section, the use of a 6 in. (15.2 cm) stabilized soil layer greatly improved the structural response of the track support system. However, no appreciable difference in response could be determined between a section with a 6 in. (15.2 cm) stabilized soil layer and a section with a 12 in. (30.5 cm) stabilized soil layer as can be seen in Figure 3.7. Pertinent results are summarized in Table 3.8.

3.5 Subgrade as a Variable

The subgrade is one of the most variable components of the track support system. The resilient response of fine-grained subgrade soils primarily depends on soil type, degree of saturation, volumetric water content, compaction, and stress state.

From ongoing research work being conducted at the University of Illinois typical average, upper bound and lower bound resilient response curves for fine-grained soils were obtained and are shown in Figure 3.8. The upper bound response corresponds generally to stiffer (stronger) soils and the lower bound response corresponds generally to softer (weaker) soils. The details of the analysis are presented below:

Section 1: Softer Subgrade Soil (See Figure 3.8)

Section 2: Average Subgrade Soil (See Figure 3.8) - reference section

Section 3: Stiffer subgrade soil (See Figure 3.8)

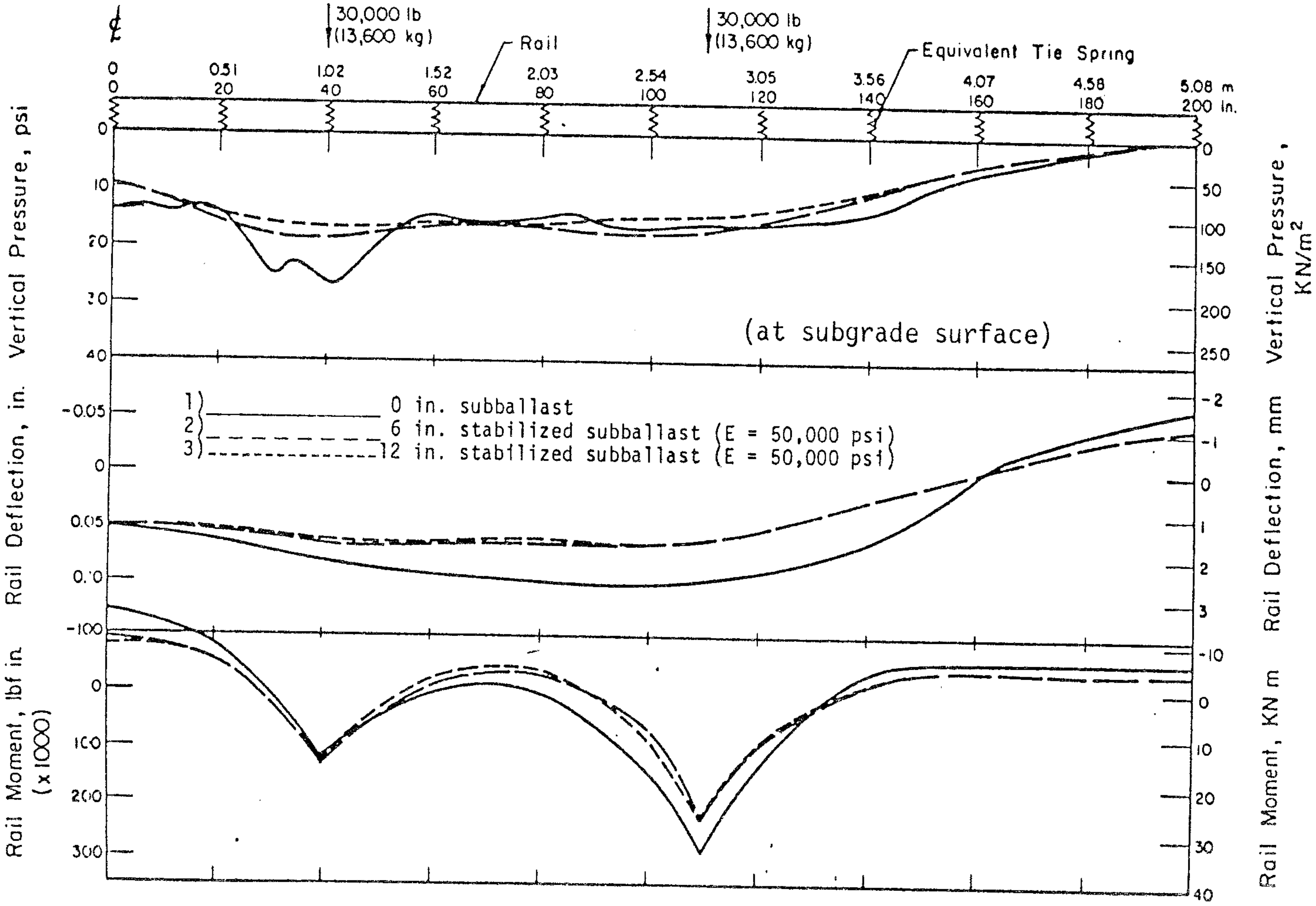


Figure 3.7 Comparison of Sections with Subballast Depth as the Variable.

Table 3.8 Comparison of Sections with Subballast Depth as the Variable

	Subballast Depth (E = 50 ksi)		
	None	6 in.	12 in.
Maximum Rail Deflection, in.	0.10	0.07	0.07
(mm)	2.5	1.8	1.8
Maximum Rail Moment, lbf in. (x1000)	297	240	245
(kNm)	33.6	27.1	27.7
Maximum Ballast Vertical Stress, psi	114.3	86.4	88.2
(kN/m^2)	788.1	595.7	608.1
Maximum Subgrade Vertical Stress, psi	26.4	18.8	16.5
(kN/m^2)	182.0	129.6	113.8
Maximum Subgrade Vertical Strain ($x10^{-4}$)	11.5	7.1	5.7

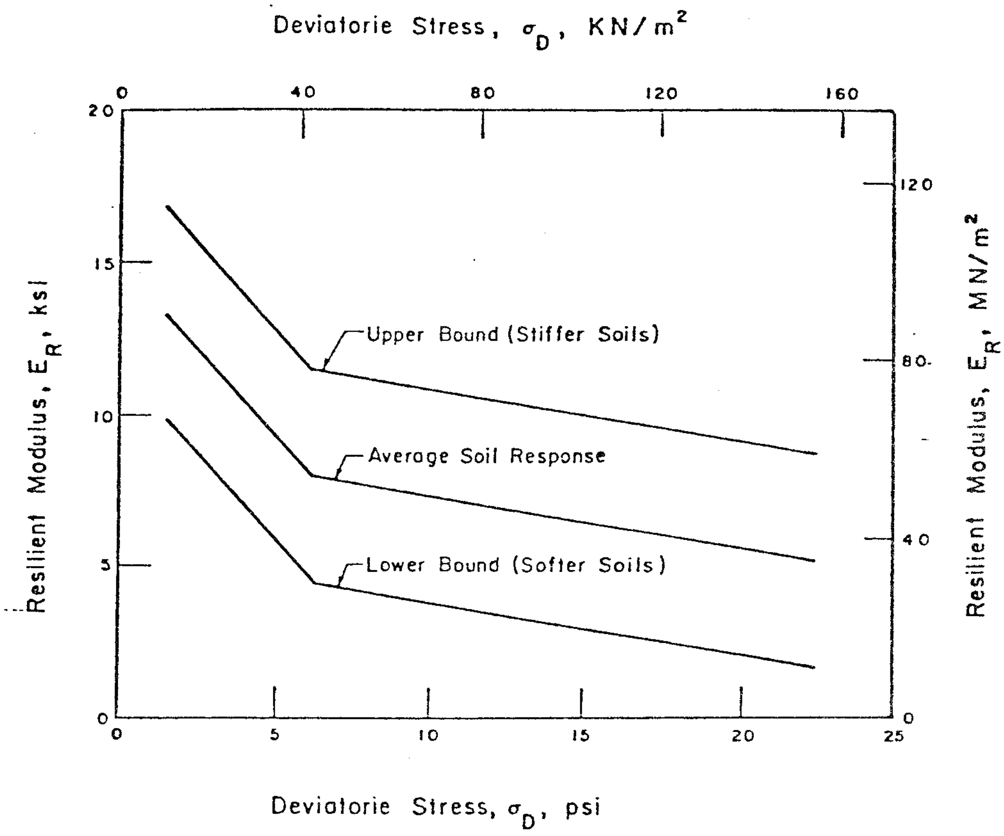


Figure 3.8 Bounds for the Resilient Response Curves for Fine-Grained Soils Used in the Parameter Study (Ref. 3).

The effect of subgrade stiffness is shown in Figure 3.9. Comparison of the rail deflection profiles for the three subgrade soils indicates that the subgrade resilient response has a substantial influence on rail deflection. While the vertical subgrade stresses tend to be similar in all the three cases, the softer subgrade will also have lower shear strength and lower resistance to accumulation of permanent deformation with repeated load applications. Pertinent results are summarized in Table 3.9.

3.6 Rail as Variable

There are many types of rails currently in use in the United States. Rail weights usually range from 115 lb/yd (57 kg/m) for lines with light traffic density to 140 lb/yd (70 kg/m) for lines with heavy traffic density. For this parameter study the following three types of rail were chosen:

Section 1: 115 lb/yd (57 kg/m) rail, $I = 65.5$ in^4 (2730 cm^4)
Section 2: 132 lb/yd (66 kg/m) rail, $I = 88.2$ in^4 (3671 cm^4)
Section 3: 136 lb/yd (68 kg/m) rail, $I = 94.7$ in^4 (3950 cm^4)-reference section

The effect of rail type is shown in Figure 3.10. The response of the track support system is almost similar for the 132 lb/yd (66 kg/m) rail and the 136 lb/yd (68 kg/m) rail. The maximum rail deflection and the maximum rail moment for the 115 lb/yd (57 kg/m) rail are slightly lower than that for the 132 lb/yd (66 kg/m) and the 136 lb/yd (69 kg/m) rails. This probably indicates that for a well maintained track, rail type has minimal influence on the transient response of the track support system. Pertinent results are summarized in Table 3.10.

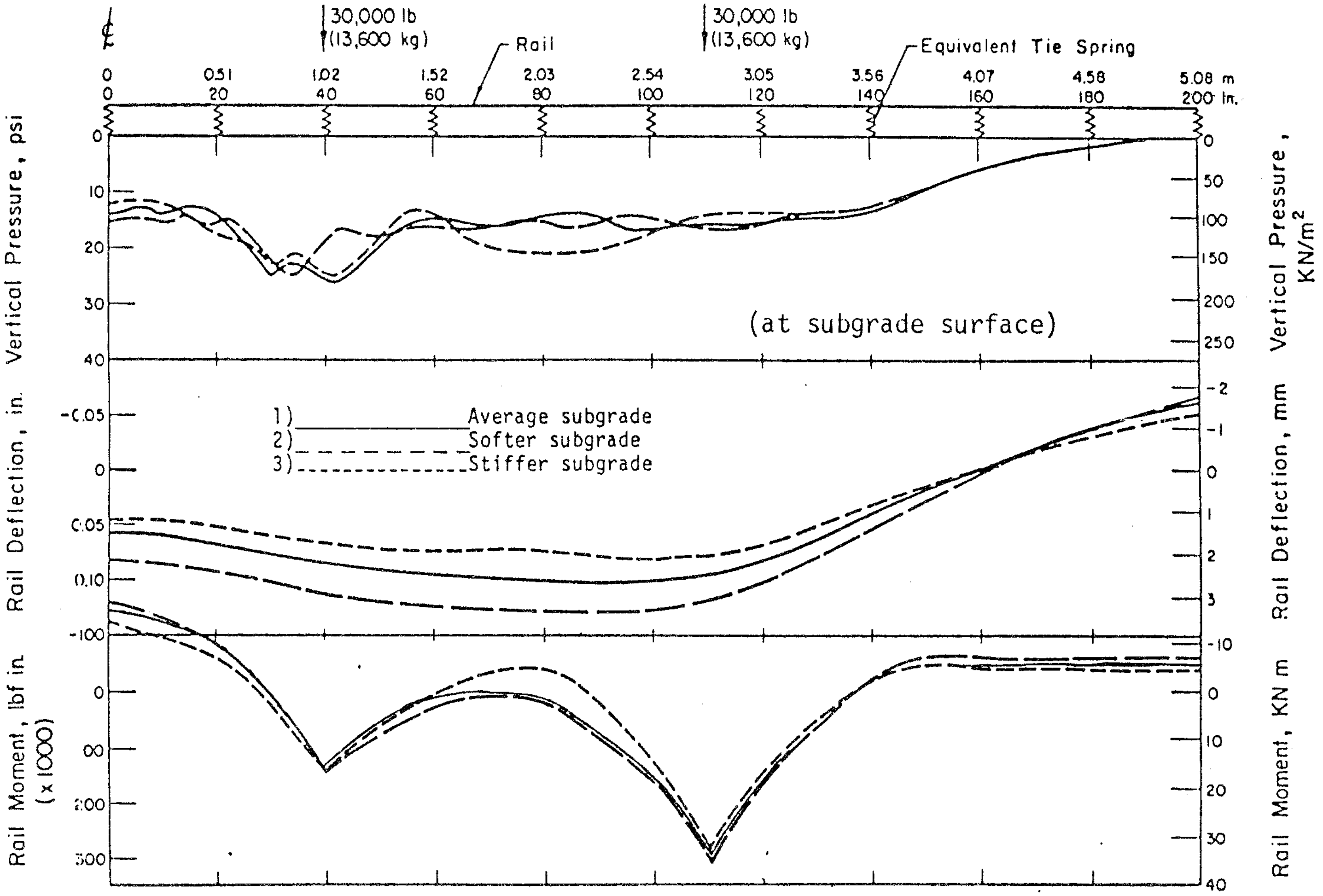


Figure 3.9 Comparison of Sections with Subgrade as the Variable

Table 3.9 Comparison of Sections with Subgrade as the Variable

	Subgrade Type		
	Softer Subgrade	Average Subgrade	Stiffer Subgrade
Maximum Rail Deflection, in.	0.12	0.10	0.08
(mm)	3.0	2.5	2.0
Maximum Rail Moment, lbf in. (x1000)	309	297	287
(kNm)	34.9	33.6	32.4
Maximum Ballast Vertical Stress, psi	92.4	114.3	105.2
(kN/m^2)	637.1	788.1	725.4
Maximum Subgrade Vertical Stress, psi	24.4	26.4	24.8
(kN/m^2)	168.2	182.0	171.0
Maximum Subgrade Vertical Strain ($x10^{-4}$)	13.0	11.5	8.0

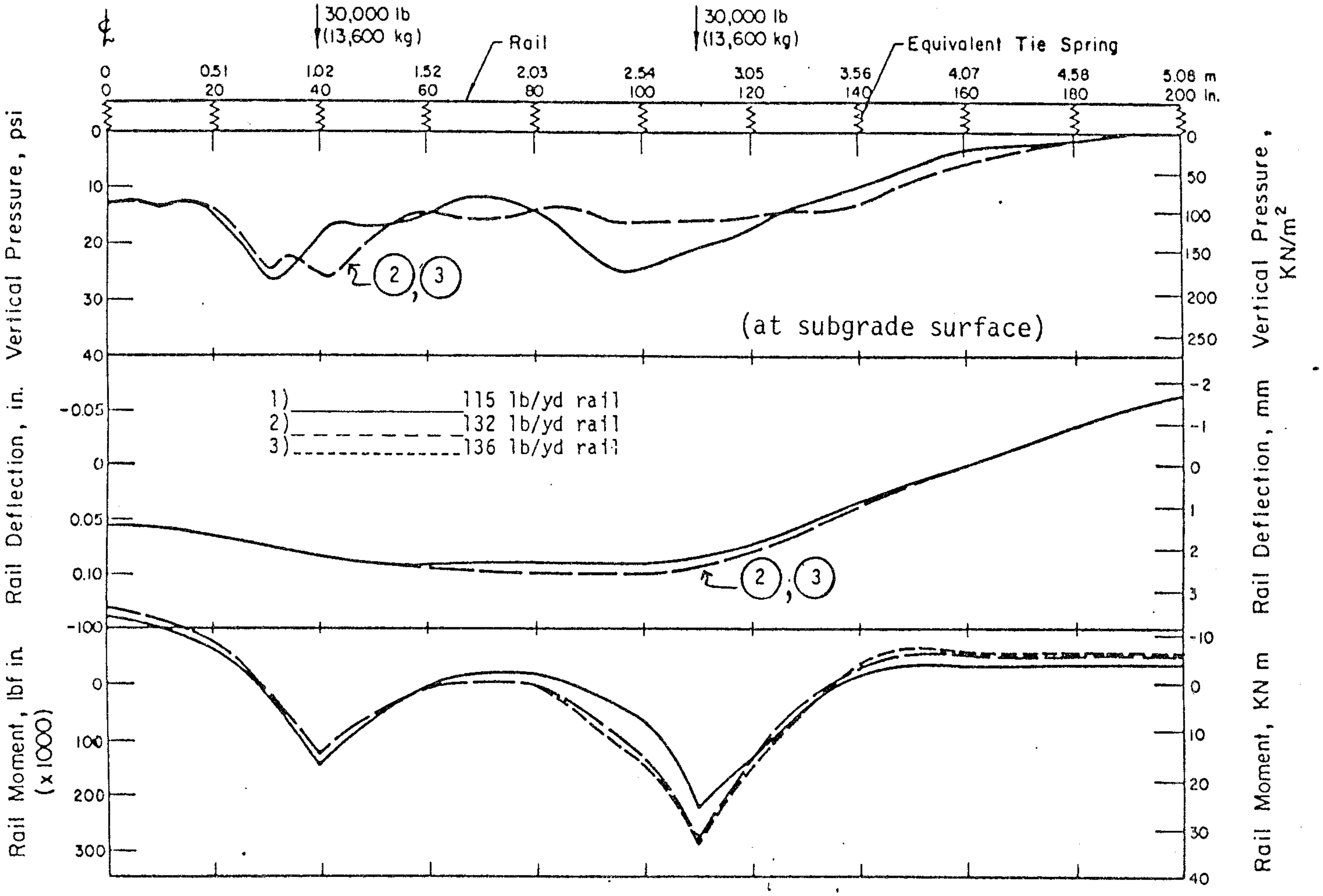


Figure 3.10 Comparison of Sections with Rail Type as the Variable.

Table 3.10 Comparison of Sections with Rail Type as the Variable

	Rail Type (lb/yd)		
	115	132	136
Maximum Rail Deflection, in.	0.09	0.10	0.10
(mm)	2.3	2.5	2.5
Maximum Rail Moment, lbf in. (x1000)	227	281	297
(kNm)	25.6	31.7	33.6
Maximum Ballast Vertical Stress, psi	94.2	114.8	114.3
(kN/m^2)	649.5	791.5	788.1
Maximum Subgrade Vertical Stress, psi	26.4	26.6	26.4
(kN/m^2)	182.0	183.4	182.0
Maximum Subgrade Vertical Strain ($x10^{-4}$)	9.4	11.6	11.5

The maximum bending stresses occurring in the rails are detailed below:

	Maximum Bending Stress			
	Compressive		Tensile	
	ksi	MN/m^2	ksi	MN/m^2
115 lb/yd rail	12.6	86.9	10.3	71.0
132 lb/yd rail	12.5	86.2	10.2	70.3
136 lb/yd rail	12.4	85.5	10.5	72.4

3.7 Tie as Variable

Two tie factors were considered, namely, tie spacing and tie base width.

3.7.1 Tie Spacing

Normal tie spacing for conventional track support systems in the United States range from 20 in. (50.8 cm) to 24 in. (61.0 cm). For this study the following tie spacings were considered:

Section 1: Tie spacing = 20 in. (50.8 cm) - reference section

Section 2: Tie spacing = 24 in. (61.0 cm)

Section 3: Tie spacing = 30 in. (76.2 cm)

The response of the three tracks to the reference loading of 30,000 lb. (13608 kg) wheel loads is given in Figure 3.11 and some of the pertinent results are summarized in Table 3.11. The effect of tie spacing is two-fold. Smaller tie spacing leads to increased overlapping effects of adjacent ties in the ballast and the subgrade, but smaller tie reactions at each tie. Increasing the tie spacing leads to decreased overlapping effects in the ballast and the subgrade, but larger tie reactions. Thus at smaller tie spacing, the overlapping effects of adjacent ties dominate while at larger tie spacing, the individual tie reaction effects dominate the outcome of the response under the ties.

This is evident from Figure 3.11 and Table 3.11. It can be seen from Table 3.11 that the response using the 24 in. (61.1 cm) tie spacing is

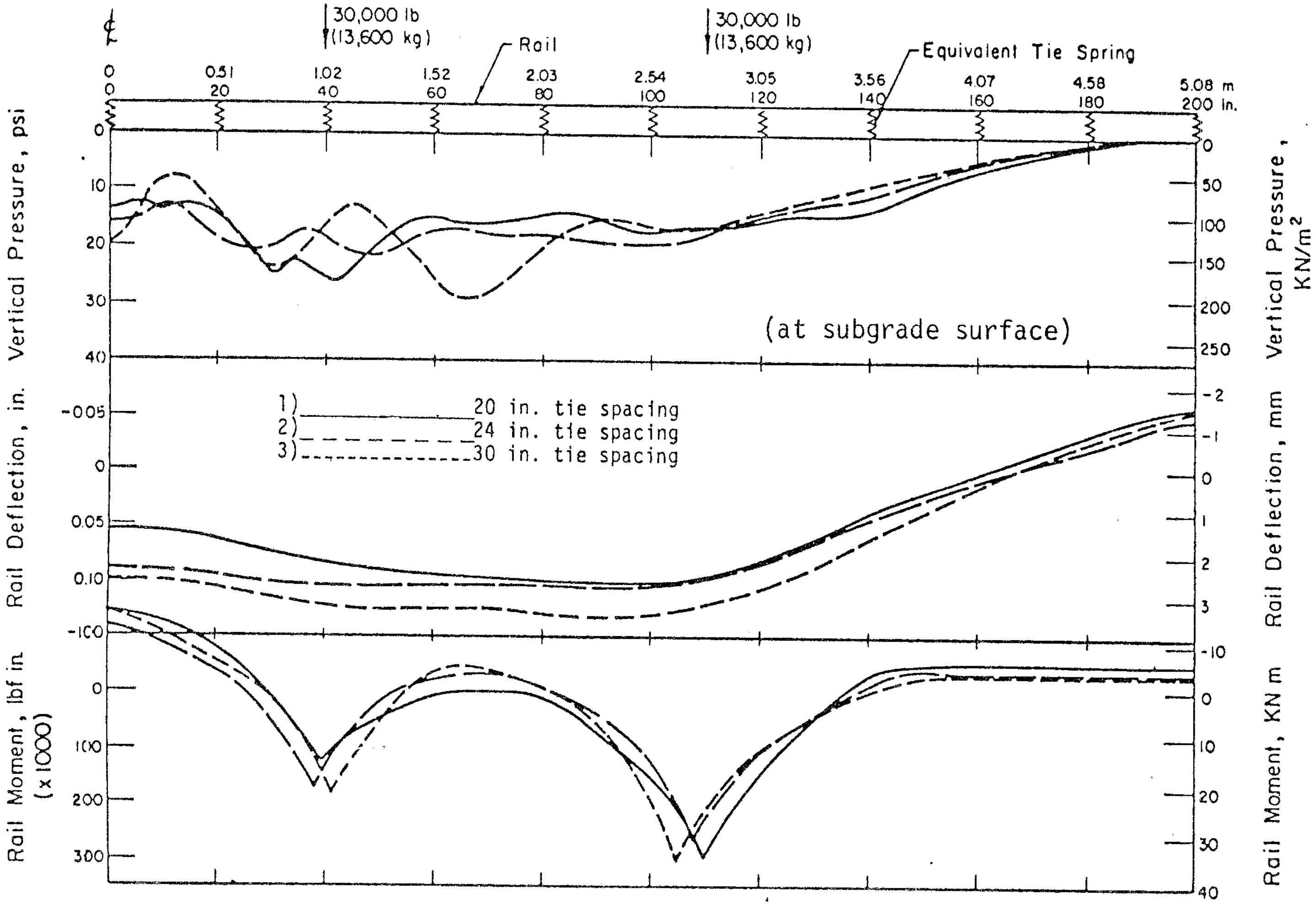


Figure 3.11 Comparison of Sections with Tie Spacing as the Variable

Table 3.11 Comparison of Sections with Tie Spacing as the Variable

	Tie Spacing (in.)				
				Finer Mesh	
	20	24	30	24	30
Maximum Rail Deflection, in.	0.10	0.11	0.13	0.09	0.12
(mm)	2.5	2.8	3.3	2.3	3.1
Maximum Rail Moment, lbf in. (x1000)	297	269	306	262	306
(kNm)	33.6	30.4	34.6	29.6	34.6
Maximum Ballast Vertical Stress, psi	114.3	87.0	122.2	90.7	121.4
(kN/m^2)	788.1	599.9	842.6	625.4	837.0
Maximum Subgrade Vertical Stress, psi	26.4	21.9	28.1	21.2	28.2
(kN/m^2)	182.0	151.0	193.7	146.2	194.4
Maximum Subgrade Vertical Strain ($x10^{-4}$)	11.5	10.8	14.0	9.4	13.8

better than that using the 20 in. (50.8 cm) tie spacing. To verify the results, computer runs were made with finer mesh for the 24 in. (61.0 cm) and the 30 in. (76.2 cm) tie spacings and the pertinent results are shown in Table 3.11. From this analysis, it appears that there is an optimum tie spacing between 20 in. (50.8 cm) and 30 in. (76.2 cm) in which case neither the overlapping effect of adjacent ties nor the effect of individual tie reaction dominate.

3.7.2 Tie Base Width

Wood tie width range from 6 in. (15.2 cm) wide on top for AREA (American Railway Engineering Association) Size 1 to 9 in. (22.8 cm) wide on top for AREA Size 5 (Ref. 4). For the purpose of this study the following three tie base widths were considered:

Section 1: Tie base width = 8 in. (20.3 cm) - reference section
Section 2: Tie base width = 10 in. (25.4 cm)
Section 3: Tie base width = 12 in. (30.5 cm)

The maximum deflection and the maximum rail moment are similar for the three sections. There is a reduction in maximum ballast vertical stress and in maximum subgrade vertical strain with an increase in tie base width. Pertinent results are summarized in Table 3.12.

3.8 Loading as Variable

Four different wheel loads were considered. The heavier wheel loads were included to simulate different degrees of impact type loading.

Section 1: Wheel load = 20,000 lb (9072 kg)
Section 2: Wheel load = 30,000 lb (13608 kg)
Section 3: Wheel load = 60,000 lb (27216 kg)
Section 4: Wheel load = 80,000 lb (36287 kg)

The response of the reference track support system subjected to the above detailed wheel loading is shown in Figure 3.12, and pertinent results are summarized in Table 3.13.

Table 3.12 Comparison of Sections with Tie Base Width as the Variable

	Tie Base Width (in.)		
	8	10	12
Maximum Rail Deflection, in.	0.10	0.10	0.10
(mm)	2.5	2.5	2.5
Maximum Rail Moment, lbf in. (x1000)	297	296	299
(kNm)	33.6	33.4	33.8
Maximum Ballast Vertical Stress, psi	114.3	82.5	65.6
(kN/m^2)	788.1	568.8	452.3
Maximum Subgrade Vertical Stress, psi	26.4	24.8	24.9
(kN/m^2)	182.0	171.0	171.7
Maximum Subgrade Vertical Strain ($x10^{-4}$)	11.5	9.9	8.0

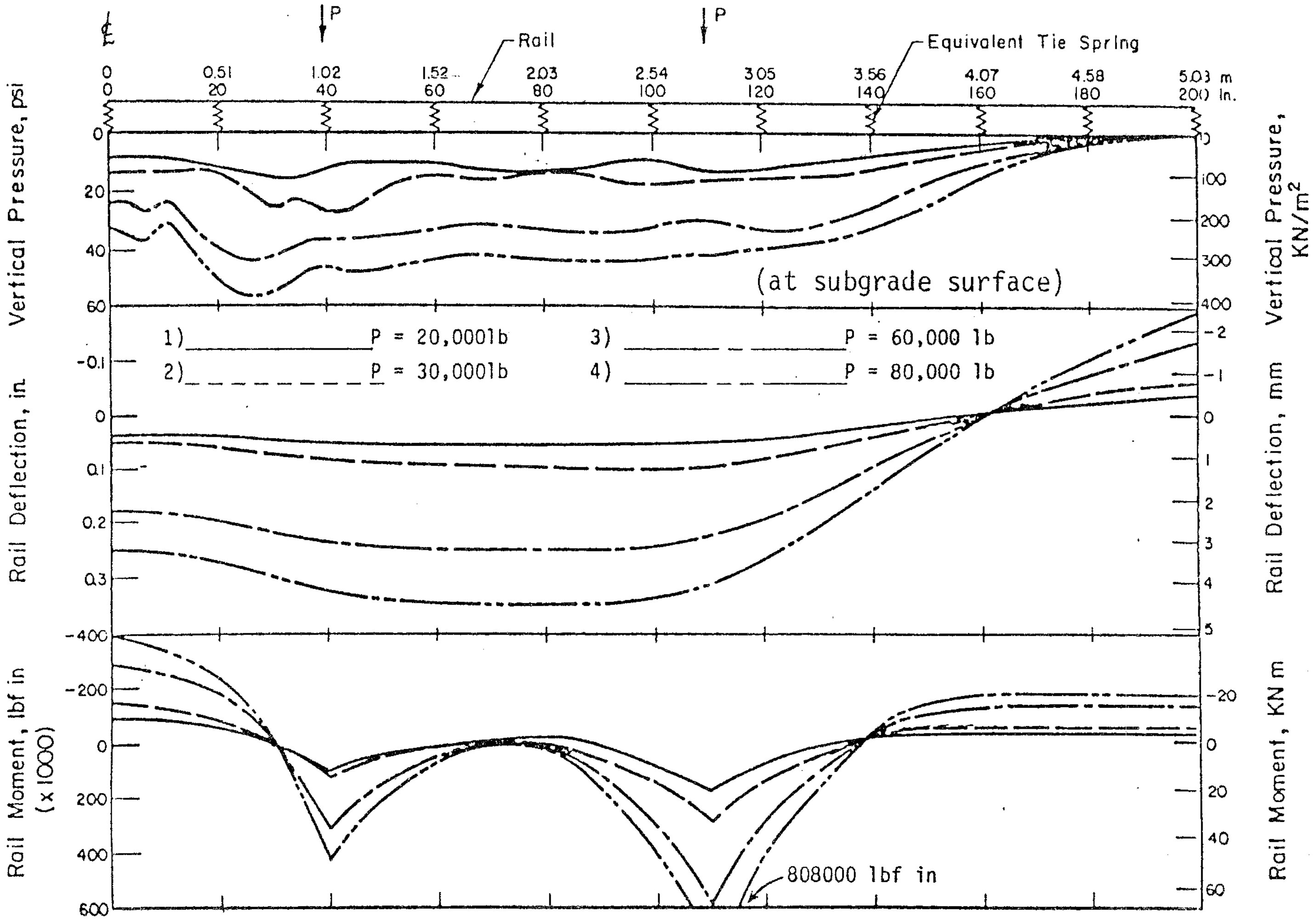


Figure 3.12 Comparison of Sections with Wheel Load as the Variable

Table 3.13 Comparison of Sections with Wheel Load as the Variable

	Wheel Load (lb)			
	20000	30000	60000	80000
Maximum Rail Deflection, in.	0.06	0.10	0.25	0.35
(mm)	1.5	2.5	6.4	8.9
Maximum Rail Moment, lbf in. (x1000)	179	297	596	808
(kNm)	20.2	33.6	67.3	91.3
Maximum Ballast Vertical Stress, psi	51.9	114.3	130.5	175.5
(kN/m^2)	357.9	788.1	899.8	1210.1
Maximum Subgrade Vertical Stress, psi	15.5	26.4	44.2	55.9
(kN/m^2)	106.8	182.0	304.8	385.4
Maximum Subgrade Vertical Strain ($x10^{-4}$)	4.8	11.5	20.0	27.9

An increase in wheel loads leads to an increasingly detrimental response of the track support system. The magnitude of the detrimental response can be seen in Figure 3.12 and Table 3.13. For example, increasing the wheel load from 20,000 lb (9072 kg) to 60,000 lb (27216 kg) increases the maximum rail deflection and the maximum subgrade vertical strain by more than 4 times and the maximum rail moment and the maximum subgrade vertical stress by about 3 times. The increase in rail moment with increased loading is significant because it can lead to earlier rail failures due to fatigue. The maximum rail bending stresses occurring for the different wheel loads are detailed below:

	Maximum Bending Stress in the Rail (136 lb/yd rail)			
	Compressive		Tensile	
Wheel Load (lb)	ksi	MN/m^2	ksi	MN/m^2
20,000	7.5	51.7	6.3	43.4
30,000	12.4	85.5	10.5	72.4
60,000	24.9	171.7	21.1	145.5
80,000	33.8	233.1	28.6	197.2

3.9 Missing Ties

The effects of missing ties or "hanging ties" on the response of the track support system were considered as follows:

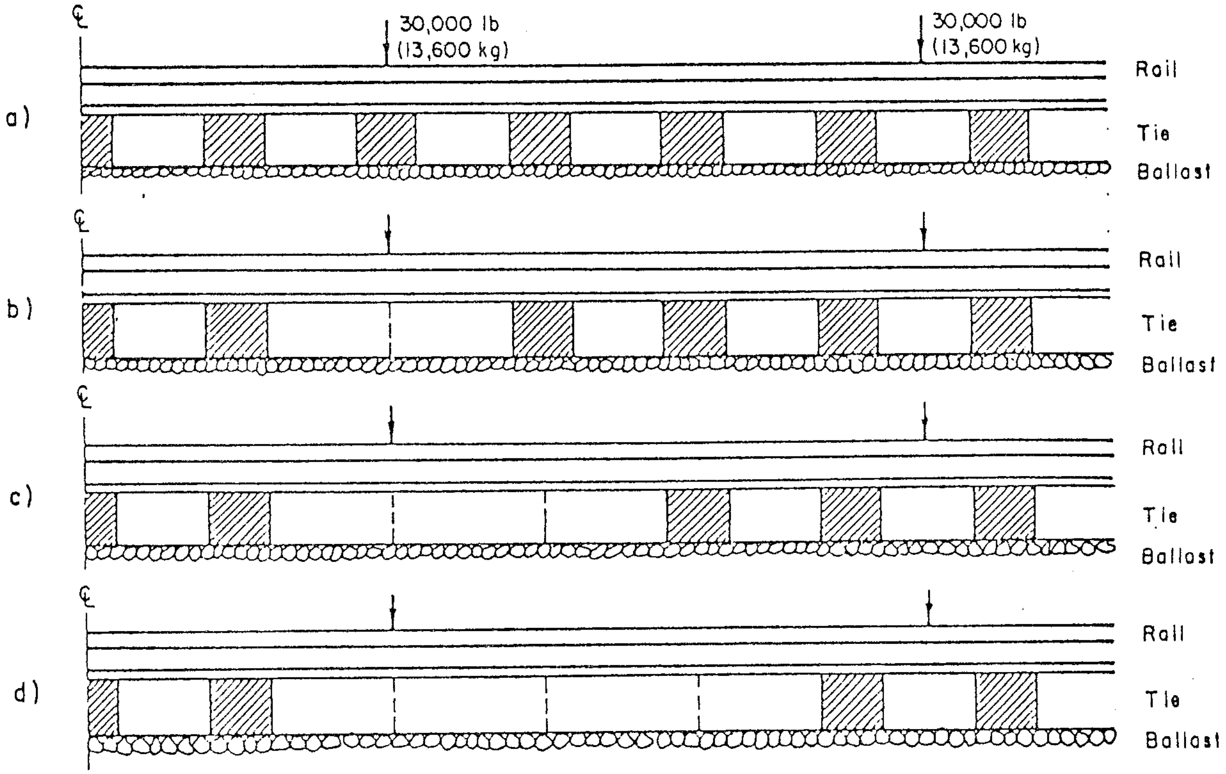
Section 1: No ties missing - reference section

Section 2: One tie missing in a row

Section 3: Two ties missing in a row

Section 4: Three ties missing in a row

The above sections are detailed in Figure 3.13. Figure 3.14 shows the deflection profile of the ballast surface relative to the deflection profile of the rail, demonstrating the detrimental effects of



a) No missing ties
b) One tie missing
c) Two ties missing
d) Three ties missing

Figure 3.13 Details of Sections with Missing Ties

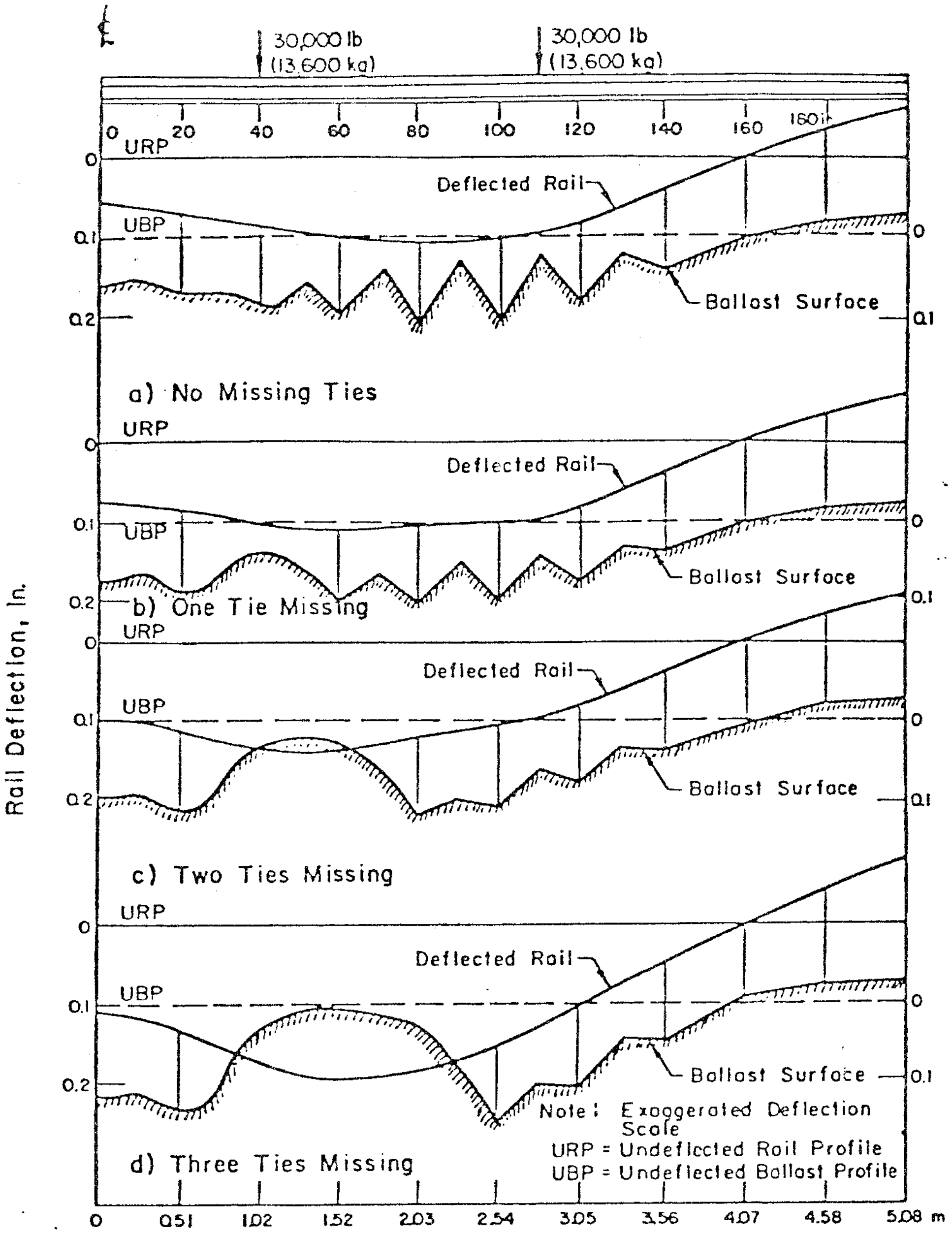


Figure 3.14 Comparison of Sections with Missing Ties as the Variable.

missing ties. The relative displacement of ballast particles in the vicinity of missing ties as compared to that under adjacent ties is very large. In a normal track section without any missing ties, this relative displacement of ballast particles does exist as shown in Figure 3.14. The relative displacement is greatly accentuated when missing ties exist. The significance of the relative displacement of ballast particles can be realized if the permanent deformation characteristics of the open graded ballast aggregate matrix are considered. The overall strain of an aggregate mass is the result of deformation of individual particles and the result of relative sliding between particles. In the case of open graded aggregate matrix, the latter part of the strain tends to dominate especially at higher σ_1/σ_3 stress ratios. The relative sliding between aggregate particle is largely irreversible and thus the deflection profile of the ballast surface shown in Figure 3.14, that develops due to missing ties can lead to a loss of alignment in the ballast surface at a quicker rate resulting in poorly performing track.

Pertinent results of the analysis of missing ties are summarized in Table 3.14. It is seen that increasing number of adjacent missing ties result in increased maximum rail deflection, increased maximum subgrade vertical strain and increased maximum tie reaction. Also a change occurs in the pattern of rail moment.

3.10 Type of Tie

Two types of ties were considered.

Section 1: Wood tie-7 in. (17.8 cm) thick x 8 in. (20.3 cm) wide x 96 in. (2.44 m)

Section 2: Concrete tie - 7 in. (17.8 cm) thick x 12 in. (30.5 cm) wide x 102 in. (2.59 m)

Table 3.14 Comparison of Sections with Missing Ties as the Variable

	No. of Missing Ties			
	0	1	2	3
Maximum Rail Deflection, in.	0.10	0.11	0.13	0.19
(mm)	2.5	2.8	3.3	4.8
Maximum Rail Moment, lbf in. (x1000)	297	273	270[1]	265[1]
(kNm)	33.6	30.8	30.5	29.9
Maximum Ballast Vertical Stress, psi	114.3	73.5	76.5	99.2
(kN/m^2)	788.1	506.8	527.5	684.0
Maximum Subgrade Vertical Stress, psi	26.4	23.1	28.1	31.3
(kN/m^2)	182.0	159.3	193.7	215.8
Maximum Subgrade Vertical Strain ($x10^{-4}$)	11.5	11.1	17.0	23.8

1 - Occurs under first load.

The wood tie was assumed to have a modulus of elasticity of 1,250 ksi (8618 MN/m^2) and the concrete tie was assumed to have a modulus of elasticity of 3,000 ksi (20684 MN/m^2). A transverse analysis was conducted for each section using a constant deflection input of 0.1025 in. (2.6 mm) as given from the longitudinal analysis at the intersection of the rail and the tie. Figure 3.15 compares tie moment, tie deflection and vertical ballast and subgrade pressures for the two ties. While the deflection profiles are similar for the two ties, there is a difference in the bending moments. The wood tie has a maximum bending moment of 79,000 lbf in. (8.93 KNm), while the concrete tie has a maximum bending moment of 176,000 lbf in. (19.88 kNm). Also the distribution of ballast/tie interface pressure is different. The distribution of the vertical pressure at the surface of the subgrade remains almost the same. It should be noted that for the above study a constant tie deflection value was used as input. In a concrete tie section, the maximum tie deflection obtained from the longitudinal analysis would be different especially if different tie spacings were used.

3.11 Discussion

1) For the parameter study, it was assumed that all the sections considered were properly maintained sections, that is to say, there were no gaps between rail and tie or tie and ballast. Firm seating was assumed of the rail on tie and tie on ballast and this must be kept in mind when interpreting any of the results of the parameter study.

2) Basically, the stiffness of the CRTSS (conventional railway track support system) is derived from two sources - the rail subsystem and the foundation subsystem which includes the ballast, the subballast, and the subgrade. When the stiffness of the rail subsystem is high as in the case of a 136 lb/yd (68 kg/m) rail, the variability in the stiffness of

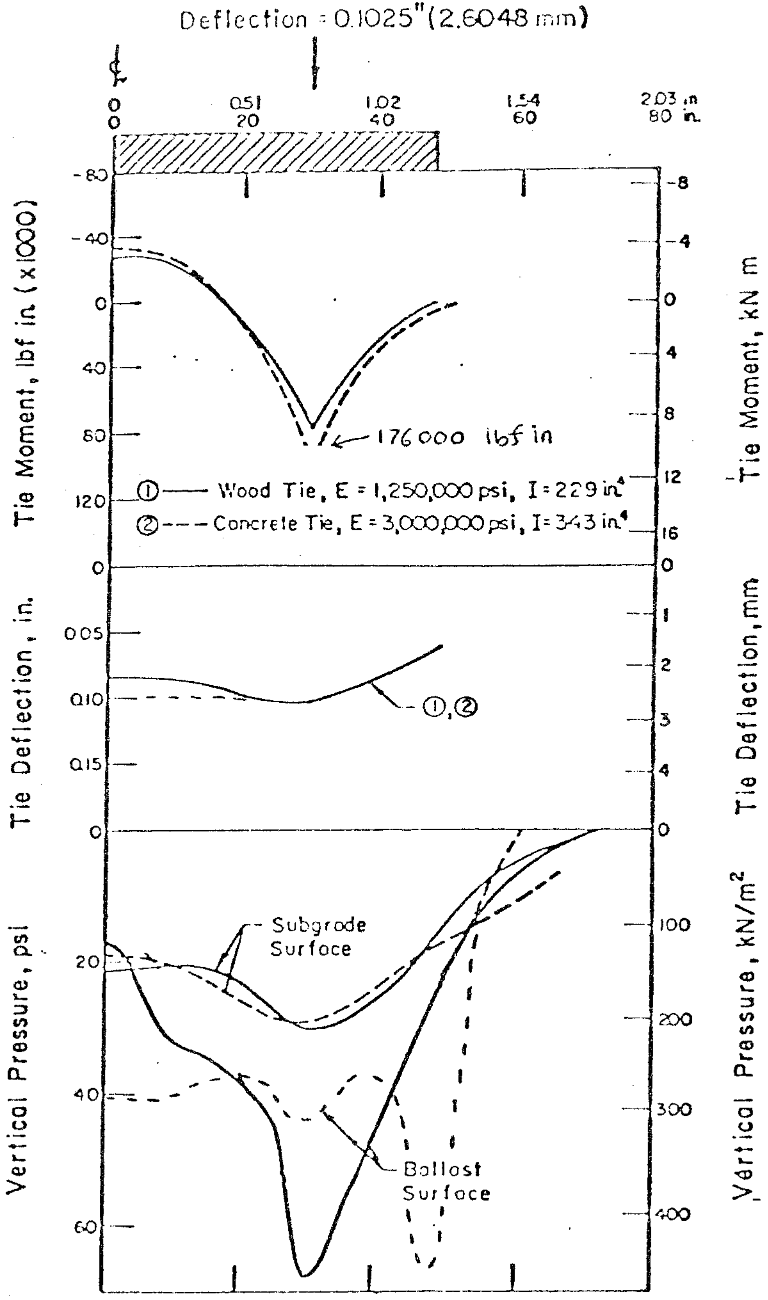


Figure 5.15. Comparison of Sections with Tie Type as the Variable.

the foundation subsystem has less influence on the response of the CRTSS than for a less stiffer rail (e.g., 115/lb/yd (57 kg/m) rail). Thus, for a poorly maintained track with substantial foundation subsystem variability and poorly maintained ties the use of a stiffer rail might be beneficial. Also a more stiffer rail might be more beneficial for lateral stability considerations.

3) The transient response of the CRTSS is not very dependent on the type of ballast used as shown in Figure 3.4. Laboratory testing has shown that the E_R-θ resilient response curves for most of the ballasts lie in a very narrow band (Ref. 5). Thus, it appears that for evaluation of transient response of CRTSS, standardized E_R-θ resilient response curves for the various ballast types may be used in analyzing the transient response of the CRTSS.

However, the long term behavior of ballast under repeated (traffic) loading and changing environmental conditions is significantly dependent on ballast type, and this should be considered when evaluating different ballast types.

4) The effect of a variable foundation subsystem can be reduced by using a stabilized subballast. The stabilized subballast aids in distributing the load more uniformly on the subgrade and maintains the ballast aggregate matrix in a more confined state allowing the ballast to develop higher stiffness.

The development of stiffness at the bottom of the ballast layer is very much dependent on the stiffness of the layer under it (Ref. 6). When the ratio of the moduli values of the ballast layer and the layer under it is below a certain value, horizontal compressive stress develops at the bottom of the ballast layer. When that ratio is above a certain value, horizontal tensile stress develops at the bottom of the ballast layer. With the use of a stabilized layer as discussed in Section 3.4, a very low modular ratio can be maintained resulting in horizontal compressive stress at the bottom of the ballast layer. Thus, the ballast layer can develop higher stiffness and the response of a CRTSS with a stabilized layer is more favorable under traffic loading than that of a CRTSS without a stabilized layer.

5) One of the most variable components in the CRTSS in the subgrade. Variation in the subgrade support can be a result of soil type, moisture content, frost action, compaction conditions, etc. As shown in Figure 3.8, the variation in the strength of the subgrade soils was found to be one of the most important parameters influencing the response of the CRTSS. Thus, on a given track section with non-uniform (in terms of stiffness) subgrade the response due to traffic loading can be very erratic. The desirability for uniform and stable subgrades is apparent.

6) The results of the parameter study indicate that rail type and tie base width has little influence on the system response of the CRTSS. Increase in tie base width does result in reduction in maximum ballast vertical pressure and maximum subgrade strain. On the other hand, increasing tie spacing leads to a detrimental response in terms of maximum rail deflection and pattern of subgrade vertical stress. Increased tie spacing leads to localized concentration of stress on the subgrade (below the ties).

7) Over the years the tracks in the United States have been deteriorating due to increased traffic frequency, heavier wheel loads and inadequate maintenance of the CRTSS. As discussed in Section 3.8, increased wheel loading leads to an increasingly detrimental response of

the CRTSS resulting in an early failure of the CRTSS. When increased wheel loading is anticipated on a given line, it is necessary to evaluate the CRTSS to ensure that the response patterns in all of the components are acceptable.

CHAPTER 4

SUMMARY AND CONCLUSIONS

4.1 Summary

1) Before serious considerations can be given to the structural analysis of conventional railway track support system (CRTSS), it must be borne in mind that the CRTSS is not composed only of rails and ties. A large portion of the structural strength is derived from the ballast, the subballast, and the subgrade, i.e., the ballast, the subballast, and the subgrade also act as load carrying media. Like any other structural media the ballast, the subballast, and the subgrade have limiting (or allowable) response patterns. Therefore any analysis of the CRTSS should incorporate the evaluation of the response patterns within the ballast, the subballast, and the subgrade.

The ILLI-TRACK finite element model of the CRTSS represents the first attempt to adequately characterize the transient response of the CRTSS to applied vertical loading by considering significant engineering properties and/or characteristics of the CRTSS components with particular attention to the ballast, the subballast, and the subgrade.

2) The mechanistic characterization of the ballast and the subgrade has been achieved using the results of repeated load triaxial tests. However, the open-graded nature of the ballast aggregate matrix, when considered as part of the CRTSS, does not lend itself to proper simulation in the structural model because the ballast in crib and shoulder areas is in a free state, that is, it is free to displace unrestricted in at least one direction when subjected to loading. In a confined state, ballast has a potential for developing very high stiffness, but in a free state the ballast aggregate matrix can generate very little resistance to loading.

The finite element model results indicate that tensile stresses may exist in the ballast material. The resistance to the tensile stress would depend on the dead load stresses and granular interlock which is a function of the compactive effort used, "age", and location of the ballast particles. In the finite element model analysis, when any ballast element shows a tensile stress, it is assigned a comparatively lower modulus value to account for the corresponding loss in stiffness.

3) The finite element model should be considered as an input to a larger model or system that would be able to predict the performance of the CRTSS. Since performance is evaluated with respect to the ability of the CRTSS to fulfill its functional requirements, it is essential to establish performance criteria for the whole system as well as for each subsystem.

4) Essentially the CRTSS is analogous to a series network. It is as strong and durable as its weakest component. Therefore, during design and construction, adequate consideration should be given to all the components of the CRTSS.

4.2 Conclusions

Conclusions derived from the results of this study are as follows:

1) Considering the developmental state of material characterization procedures and the lack of availability of pertinent field response data, it is felt that the finite element model of the CRTSS adequately characterizes the transient response of the CRTSS when subjected to vertical loading.

2) It is realized that there are a large number of conditions that exist in an actual CRTSS and that it would be impractical to attempt to satisfy all these conditions in a theoretical model. In certain areas,

the effect of some of the conditions can be evaluated using the results given by the finite element model and incorporating them with judgement and experience.

3) One of the most critical design factors seems to be the ballast/ subgrade interface. Ballast laid directly on top of low strength subgrade can be detrimental to satisfactory performance of the CRTSS. The desirability of a stiff layer (e.g. a stabilized subballast) between the ballast and the subgrade has been demonstrated in this study.

4) When increased traffic loading (larger wheel loads and/or increased volume) is anticipated on any line, it is necessary to evaluate the CRTSS to ensure that the anticipated performance will be achieved. The ILLI-TRACK model can be used to evaluate the structural adequacy of the present CRTSS and also the effectiveness of various proposed CRTSS structural upgrade schemes.

5) During material testing, analysis, and design phase of the CRTSS particular attention should be directed to the ballast, the subballast, and the subgrade materials.

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