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TECHNICAL DATA BASES REPORT

BALLAST AND FOUNDATION MATERIALS RESEARCH PROGRAM

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SUMMARY REPORT

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16. Abstract

Literature and other information sources pertaining to properties of granular materials, ballast and sub-ballast materials, fine-grained soils, and structural behavior models were reviewed; presented in this report is a summary of the current technology relative to the following:

1. Procedures for evaluating ballast and subgrade material properties.
2. Factors which influence ballast and soil material porperties.
3. Relations between ballast and soil material properties and track system behavior and performance
4. Applicability of structural analysis models to predicting behavior of track system.
5. Transfer functions relating track behavior to performance.

Based on the review, it is concluded that ballast and subgrade materials receive inadequate considerations in analysis and design, more realistic models are available for structural analysis of track systems, transfer functions relating track behavior to performance are not available and climatic factors exert a significant influence on behavior and performance and must be given appropriate consideration in analysis and design of the track system.

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PREFACE

This report has been generated 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 is one of the first outputs from the study and essentially describes a state of the knowledge on ballast and foundation materials.

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 Mr. W. B. O'Sullivan as the Contracting Officer's Technical Representative of the FRA on the entire research program.

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CHAPTER 1

INTRODUCTION

THE PROBLEM

Track stability is essential for economical maintenance and construction. Trains impose various types of loads (vertical, lateral, longitudinal) on the track structure and the track system deforms. Additional loads and stresses can arise through a combination of these - for example, warp, which arises from differential cross levels at separate locations within the span of a set of car trucks on opposite rails. The track structure also experiences vibration from several sources - compression and release as wheel loads pass, "bounce" imparted by spring action in the car trucks, effects of rail joint location vs car truck center spacing, impact from irregularities in track and/or equipment, and the frequency and duration of time within which these effects are produced. Although the track deformation is primarily elastic in nature, permanent deformation may also accrue under the action of repeated loading.

An adequate engineering analysis of track system response and performance involves the consideration of the components of the track system; rail-tie assemblage, the ballast materials, and the subgrade soil. Obvious problems arise from vertical loadings, but the effect of ballast characteristics, especially the geometric properties, on lateral and longitudinal stability are of much significance. Lateral and longitudinal forces are delivered from modern heavy wheel loads. Even more significant is the need for restraint of continuous welded rail where the type of ballast and design of ballast section can be critical items. Extremely high thermal expansion and contraction forces can be found in the rails. The interactions between ballast particles, between ballast and subgrade, and between ballast and ties require full investigation.

To facilitate the engineering design of the track support system, it is essential to have available the technology necessary to:

1. Determine the response (stresses, strain, deflection) of the track support system to load;
2. Define the interactions among the various components of the track support system (track-tie, ballast, subgrade);

3. Evaluate the significant engineering properties and/or characteristics of the track support system components;
4. Develop "transfer functions" and material utilization criteria by which it is possible to predict performance (long-term behavior) based on track support system response (instantaneous behavior);
5. Assess the effects of environment (primarily moisture and temperature) on track support system response and performance.

Recent efforts to develop structural response and performance models for highway and airfield pavements have indicated that the problem is extremely complex. There is no reason to assume that track support system response and performance will be less complex.

Although the problem is complex, it has been demonstrated that with appropriate materials testing methods and analytical structural models, it is possible to accurately predict the structural response (instantaneous behavior) of a pavement subjected to transient loading. Less success has been experienced in predicting pavement performance. Performance prediction is much more complicated because of the problems associated with the adequate consideration of a) environmental effects, b) material property changes with time, c) loading spectrum, etc.

Regardless of the problems associated with predicting performance, the first step toward developing improved capabilities for considering track system performance is the simultaneous development of materials testing and evaluation procedures and a mechanistic structural model. The structural model must correctly represent the stress-strain response of the subgrade and ballast materials and also adequately simulate the behavior of the rail-tie assemblage. It is not possible to establish a good performance model without first developing a satisfactory structural response model and acceptable materials testing and evaluation procedures.

PROJECT RESEARCH OBJECTIVES

The specific objectives of this research program are the following specific tasks:

Task 1. Identify failure and other useful performance criteria for ballast and foundation materials.

Task 2: Identify relationships between loading environment and ballast and foundation material behavior.

Task 3: Identify environmental factors that influence ballast and foundation material behavior and the relative extent and importance of these.

Task 4: Identify material parameters that are meaningful for evaluating performance and test procedures for determining these properties.

Task 5: For the total track structure performance, identify relationships between ballast and foundation material behavior, and environmental factors for combinations of materials, loadings, environment and track configuration.

Task 6: Perform rank ordering of the ballast, subballast, and subgrade materials according to their performance in service.

Task 7: Identify procedures where ballast and foundation materials properties may be modified to improve material performance.

Task 8: Identify the costs associated with the use of each type of ballast material.

PROJECT WORK PLAN

The work plan outlined below describes the major phases of the project activity. The manner in which the various phases of project activity contribute to the accomplishment of the specified tasks is indicated in Table 1.1.

Phase I Technical Data Bases

The relevant literature pertaining to the pertinent properties of granular materials, ballast materials, fine-grained soils, and structural models will be reviewed. Based on the reviews and other available information sources, current technical data bases will be developed.

Phase II Development of a Structural Model and Materials Evaluation Procedures

A "mechanistic structural analysis model" will be developed and testing procedures will be established for evaluating the properties of the ballast and foundation materials needed as inputs to the structural model.

Phase III Parameter Studies and Sensitivity Analyses

The structural model developed in Phase II will be utilized to establish the effects of various parameters on the response of the railway support

Table 1.1 Work Phases Contributing to the Achievement of the Designated Tasks

Task Number \ Phase Number	I	II	III	IV	V	VI
1	X					
2	X	X				
3	X	X				
4	X	X	X	X		
5	X	X	X	X		
6	X	X	X	X		
7	X	X	X	X		X
8	X		X	X	X	X

structure. Some significant parameters to be considered will include:

1. Subgrade soil resilience properties
2. Ballast and subballast material resilience properties
3. Ballast and subballast thicknesses
4. Ballast-tie parameters
5. Rail-tie parameters
6. Environmental factors such as temperature and moisture

Phase IV Materials Evaluation Study

Based on Phase I, II, and III findings, it will be possible to identify those material (subgrade soil, ballast, subballast) properties that significantly influence track structure behavior and performance. A series of laboratory tests will be conducted with selected foundation soils and ballast and subballast materials to determine their pertinent engineering properties as previously identified. The ballast, subballast, and foundation materials selected for use in the laboratory study will represent a range in both engineering properties and types and sources of materials.

Phase V Economic Evaluation

Costs associated with the use of each type of ballast material will be identified and the cost effectiveness of the various ballast materials ranked both as to transportation costs and stability. In so doing, the concept of overall economic costs, C_e, will be introduced.

where: $C_e = f(x_1, x_2, x_3, x_4, \text{etc.})$ (1.1)

and x_1 = purchase price

x_2 = transportation cost

x_3 = track maintenance cost (frequency of surfacing and smoothing cycle)

x_4 = tie life with various ballast types - cost effects

x_5, x_6, etc. = other cost factors related to ballast

Phase VI Preparation of Conclusions, Summary, and Recommendations

Data and information obtained from the technical literature and that developed in the project will be summarized and analyzed with respect to establishing definitive responses to the previously described tasks. Appropriate conclusions and recommendations will be developed and areas of technological need will be identified.

REPORT OBJECTIVE

The objective of this report is to develop and summarize the status of current technology relative to the following:

1. Procedures and techniques for evaluating ballast and subgrade material properties (shear strength, resilience, susceptibility to plastic deformation, capacity for lateral and longitudinal restraint, etc.)
2. Factors which significantly influence material properties (water content, density, durability, loading conditions, gradations, etc.)
3. Relations between material properties and track system response and performance leading to an identification of the cause and development of failures and other useful performance criteria, performance indices, or material parameters required to elude failure will, where possible at this stage of effort, be identified.
4. Review and application of "structural models" such as those of Meacham, Lundgren, and others relative to predicting the behavior of the track support system (subgrade-ballast-tie-rail system).
5. Identify "transfer functions" which relate track system response, stress or strain levels in materials and subgrades, etc., to performance.

REPORT ORGANIZATION

General history, background, and other pertinent considerations are presented in Chapter 2. Chapters 3 and 4 are devoted to extensive discussion of ballast and roadbed materials. Existing methods for analysis and design of the railway support system are reviewed in Chapter 5. Chapter 6 covers the effect of the ballast and roadbed material strength properties on track support system response. In Chapter 7, various aspects and considerations relative to predicting the performance of railway support systems are discussed. Chapter 8 contains a summary and conclusions relative to the current technology for analyzing and designing railway support systems.

CHAPTER 2

THE RAILWAY SUPPORT SYSTEM

HISTORY

The first railroad track in the United States is said to have been used to haul granite for the construction of the Bunker Hill Monument in 1826. The track consisted of timber stringers laid on stone sleepers set in the subgrade. Later, about 1830, railroads replaced the timber stringers with a flat-bottomed Tee-Rail attached to timber sleepers or ties. The cross ties, obtained from timber cut along the right of way, were embedded in broken stone or gravel, also obtained along the location site. Various materials were used over the ensuing years--sand, gravel, cinders, sea shells, lumps of burnt clay, and sometimes plain soil. Track for the heaviest traffic however used crushed rock, prepared gravels, and where available, crushed slag. Which of these many types of ballast are the most stable and economical has been determined largely by experience of individual railroads--and that experience has varied.

There has been little effort to incorporate stability factors into ballast specifications. A principal reason for the use of ballast is to distribute the load to the roadbed. Providing drainage and resilience for the track-tie-structure are important ballast functions, but anchoring the track, providing vertical, lateral, and longitudinal restraint is a primary and essential function. The need for high quality ballast has become even more critical recently because of the increasing demands placed on the railway support structure by rising volumes of traffic, heavier wheel loads, higher speeds, higher centers of gravity, longer truck centers, and three axle trucks.

PERTINENT COMPONENTS OF THE RAILWAY SUPPORT SYSTEM

The conventional railway support system is made up of certain components including rails, ties, tie plates, spikes, joints, ballast, and roadbed.

Rails

Rails range in weight from 90 to 155 lb/yd (45 to 78 kg/m), although lighter sections are used on some low-density branch lines. The usual rail weights are 115 or 119 lb/yd (57 or 60 kg/m) for lines of light and medium traffic density and 132 or 136 lb/yd (66 or 68 kg/m) for lines of high traffic density. Weights of 140 lb/yd (70 kg/m) and heavier are found where traffic volumes are very heavy. Rail steel has a modulus of elasticity of 30×10^6 psi (207×10^6 kN/m^2). Chemical specifications can be found in Chapter 4, Rail, of the American Railroad Engineering Association's <u>Manual of Recommended Practice</u> (2.1). Allowable bending stress in rail is in the 30,000 to 35,000 psi range (207,000 to 241,000 kN/m^2). Other pertinent properties of rails are given in Table 2.1.

Recently, AREA (2.2) has recommended the type of rails used be limited to the following:

140 RE (RE = AREA design designation)
136 RE
132 RE
119 RE
115 RE
106 CF&I (CF & I = Colorado Fuel and Iron design designation)
100 RE
90 RA-A (RA-A = Amer. Railway Assoc. type A designation)

Wood Ties

Wood ties are in almost universal use on United States railroads. Those used in main line track, AREA grades 4 and 5, measure 7 in. (17.8 cm) deep by 8 in. (20.3 cm) wide and 7 in. (17.8 cm) deep by 9 in. (22.9 cm) wide respectively. Lengths are 8 ft. (2.44 m), 8-1/2 ft (2.59 m), or 9 ft (2.74m). The 9 ft (2.75 m) tie has been recommended by AREA for all new construction and rehabilitation. The 8-1/2 ft (2.59 m) tie is most commonly used for high-density lines. Wood ties are spaced on 18 in. (45.7 cm) to 26 in. (76.2 cm) centers but 19-1/2 in. (49.5 cm) and 22 in. (55.9 cm) (giving 24 and 22 ties per 39 ft (11.90 m) rail length respectively) are most frequently found in main tracks. Wood ties weigh 150 to 200 lb (68 to 90 kg).

Concrete Ties

Concrete ties first appeared in the United States with a 200-tie installation by the Reading Company. Various types of reinforced, monolithic

Table 2.1. Pertinent Rail Properties
(From Ref. 2.14)

Rail Size	Weight lb/yd (kg/m)	Area, A $in.^2$ (cm^2)	Moment of Inertia I_x $in.^4$ (cm^4)	I_x/A $in.^2$
BR 98	98.1 (48.7)	9.61 (62.00)	40.7 (1695)	4.24
S 49	99.6 (49.4)	9.76 (62.97)	43.7 (1819)	4.48
100 RE	101.5 (50.4)	9.95 (64.19)	49.0 (2039)	4.92
SNCF 50	102.2 (50.7)	10.00 (64.50)	48.5 (2019)	4.85
JNR 50T	107.5 (53.3)	10.52 (67.90)	54.8 (2280)	5.21
S54(IEV)/BR109	109.7 (54.4)	10.75 (69.34)	56.4 (2346)	5.25
115 RE	114.7 (56.9)	11.25 (72.58)	65.6 (2730)	5.83
119 RE	118.8 (58.9)	11.65 (75.16)	71.4 (2972)	6.13
S 60 (IEV)	121.8 (60.4)	11.91 (76.86)	73.4 (3055)	6.16
JNR 60	122.6 (60.8)	12.01 (77.50)	74.2 (3090)	6.18
S 64	130.8 (64.9)	12.82 (82.70)	78.1 (3252)	6.09
132 RE	132.1 (65.5)	12.95 (83.55)	88.2 (3671)	6.81
136 RE	136.2 (67.6)	13.35 (86.13)	94.9 (3950)	7.11

ties were tested through 1930 but experienced difficulties with fastenings, short circuits of signals, and shattering under derailments.

Interest in concrete ties revived in the 1950s as timber costs increased and European railroads experienced successful use of pre-stressed ties. The need for an alternative for the wood tie prompted the AAR to develop several designs. Of these, the Type E was adopted as having a reasonable economic parity with wood ties based on a 30 inch (76 cm) spacing.

Modifications in design to overcome distress noted in field tests led to the AAR Type 3 tie, commercially designated as the MR 3. This tie has deepened tie seats and a 25 percent prestress increase using four 7/16 in. (1.1 cm) diameter bright steel strands. These designs have also experienced distress including insert pullouts, torsional cracking in the mid-portion, broken clips and bolts, spalling of rail seat shoulders, and flexural cracking beneath the rails. Flexural cracking, originating in the bottom surface beneath the rail seat, propagated upward to the top layer of reinforcing wires. Failure of the crack to close (a major function of pre-stressing) indicated insufficient strength and bonding.

With the Portland Cement Association and American Cement Institute Committee 545 joining with the AREA Special Committee on Concrete Ties, corrective action for these weaknesses and improved specifications were developed, presented for test and discussion, and finally revised for publication in Chapter 3 of the AREA Manual (2.1).

Included in the corrective measures is a recommended reduction in prestressed transfer distance by use of a gentle release of pre-stressing and a deformed or chemically treated wire. (The use of rusted wire was suggested as an interim solution.) Flexural moment (for 30 inch (76 cm) tie spacing)was increased to a positive value of 300 kip-in. (33.9 kN-m) under the fail seat (with some allowance for negative flexural moment at that location) and tie center negative moment was increased to 200 kip-in. (22.6 kN-m) along with improved uplife requirements for the fastenings. Flexural requirements are to be revised correspondingly for tie spacings less than 30 inches (87 cm). Finally, the impact factor used in developing flexural requirements was increased from 50 to 150 percent.

An excellent discussion of the history of concrete ties may be found in Reference 2.19.

Tie Plates

For wood ties, rails are set on tie plates to reduce crushing and abrasion of ties. Plates are generally 7-3/4 in. (19.7 cm) to 8 in. (20.3 cm) wide and from 12 in. (30.5 cm) to 14 in. (35.6 cm) long; a few high-degree curves (6-degree and over) are laid with 18 in. (45.7 cm) long plates.

Spikes

Plain cut spikes 9/16 in. by 5-1/2 in. (1.4 cm by 14.0 cm), 5/8 in. by 6 in. (1.6 cm by 15.2 cm), and 5/8 in. by 6-1/2 in. (1.6 cm by 16.5 cm) are used both for railholding and for plate-holding holes in the tie plates. The last two are used with the heavier rail sections and/or special applications such as on curves.

Joints

Rails are rolled 39 ft. (11.90 m) in length (a few are rolled 78 ft. (23.8 m)) and are joined together either by bolted joints (usually 6-hole bars 36 in. (91 cm) in length but many with 4-hole bars 24 in. (61 cm) in length) or by welding into lengths of 1440 ft (439 m) which may, in turn, be field-welded into rails many thousands of feet or even miles in length. The thermal expansion and contraction forces of such long rails are restrained by rail anchors, having a friction grip on the rail base, that transmit the thermal forces to the ties.

In lieu of CWR and to reduce end batter, some roads have laid rails with tight joints, i.e., no allowance for expansion or contraction. To insure a high degree of permanence, the joint bars may be glued with an epoxy glue to the rails in addition to the use of high strength bolts. Rail anchorage similar to that required for CWR should be used with these installations. Further discussion of bonded or glued joints may be found in Reference 2.20.

Ballast, Subballast, and Roadbed

The rail-tie assemblage is normally founded upon ballast and in some cases subballast which in turn is founded on the roadbed. Each of these components has certain required characteristics. These components will be discussed more extensively in subsequent sections.

BALLAST MATERIALS

Types and Quantities of Ballast Materials Used

The types or kinds and quantities of ballast materials used by various railroads depend on a number of factors including:

1. type and amount of traffic
2. tonnage
3. types of ballast available
4. subgrade and climatic conditions
5. certain complex economic factors

Table 2.2 and 2.3 summarize the results of a questionnaire sent out by AREA Committee 1 (2.12) to various railroads to determine the types and gradations of ballast used.

Table 2.4 summarizes the quantities of various types of ballast used in the years 1971 and 1972 as reported by the Bureau of Mines (2.3). In a recent issue of MODERN RAILROADS (January, 1975) the results of a questionnaire indicated a total of 18,300,000 tons (16,600,000 metric tons) of ballast is to be placed in 1975. A number of railroads did not respond, thus the total probably will be substantially greater.

Tables 2.5, 2.6 and 2.7 summarize the recommended AREA gradations for various types of ballast materials.

Functions of Ballast

Peckover (2.4) states that "Good ballast material provides elastic support and anchorage for track with a minimum of maintenance." Functions of ballast as defined by Hay (2.5) and Peckover (2.6) are paraphrased below:

1. Transmit the imposed loadings uniformly to the roadbed at a pressure tolerable for the particular material in the roadbed.
2. Provide uniform support for the ties with the necessary degree of elasticity and resilience to absorb vibrations and shock.

Table 2.2. Types of Ballast Used
(From Ref. 2.12)

Type	No. of RR Using	Percent
Limestone	35	38.0
Slag	23	25.0
Granite	15	16.3
Trap Rock	8	8.7
Quartzite	5	5.4
Dolomite	3	3.3
Basalt	1	1.1
Rhyolite	1	1.1
Burned Shale	1	1.1
	92*	100.0
<u>Gravel</u>		
Pit Run	10	34.4
Crushed	7	24.4
Washed	3	10.2
Washed and Crushed	9	31.0
	29*	

*Actual total is less; some railroads use more than one type.

Table 2.3. Size of Ballast Used
(From Ref. 2.12)

Type	AREA Spec. No.	Size (in.)	No. of RR Using	Percent
Stone	24	2 1/2 - 3/4	9	9.8
	3	2 - 1	16	17.4
	4	1 1/2 - 3/4	40	43.5
	5	1 - 3/8	10	10.9
		2 - 3/8	2	2.1
		1 3/4 - 3/4	2	2.1
		1 3/4 - 5/8	4	4.4
		1 1/2 - 1	1	1.1
		1 1/2 - 1/2	4	4.3
		1 1/2 - 1/4	1	1.1
		1 1/4 - 1/2	2	2.2
		1 3/8 - 3/8	1	1.1
			92*	100.0
Gravel	G-1		8	27.6
	G-2		4	13.8
	G-3		7	24.1
	Pit Run		10	34.5
			29*	100.0

* Actual total is less; some railroads use more than one type.

Table 2.4. Materials Used for Railroad Ballast (From Ref. 2.3).

Material	Amount, Short tons	
	1971	1972
Limestone and Dolomite	6,153,000	7,250,000
Granite	5,388,000	6,162,000
Slag (air cooled blast furnace)	3,174,000	3,686,000
Gravel	2,347,000	2,229,000
Other Stone	1,538,000	N.A.
Trap Rock	989,000	2,332,000
Steel Slag	855,000	1,327,000
Sandstone and Quartzite	610,000	1,014,000

N.A. = Data Not Available.

Table 2.5. AREA Gradation for Crushed Stone and Crushed Slab (From Ref. 2.1).

Size No.	Nominal Size Square Opening	Amounts Finer Than Each Sieve (Square Opening) Percent By Weight									
		3"	2 1/2"	2"	1 1/2"	1"	3/4"	1/2"	3/8"	No. 4	No. 8
24	2 1/2"-3/4"	100	90-100		25-60		0-10	0-5			
3	2"-1"		100	95-100	35-70	0-15		0-5			
4	1 1/2"-3/4"			100	90-100	20-55	0-15		0-5		
5	1"-3/8"				100	90-100	40-75	15-35	0-15	0-5	
57	1"-No. 4				100	95-100		25-60		0-10	0-5

Table 2.6. AREA Gradations for Gravel
(From Ref. 2.1)

Size No.	Percent Crushed Particles	Amounts Finer Than Each Sieve (Square Opening) Percent By Weight							
		1 1/2"	1"	1/2"	No. 4	No. 8	No. 16	No. 50	No. 100
G-1	0-20	100	80-100	50-85	20-40	15-35	5-25	0-10	0-2
G-2	21-40	100	65-100	35-75	10-35	0-10	0-5		
G-3	41-75	100	60-95	25-50	0-15	0-5			

Table 2.7. Specifications For Pit-Run
Gravel Ballast (From Ref. 2.15)

Sieve Size (Square Openings)	Amounts Finer Than Each Sieve Percent By Weight	
	Grade A	Grade B
2 1/2 in.	97-100	97-100
No. 4	20-55	20-65
No. 200	0-2	0-3

3. Anchor the track in place and resist vertical, lateral and longitudinal movement.

4. Provide immediate or free drainage and prevent the growth of vegetation.

5. Resist degradation due to physical forces exerted by traffic and maintenance equipment and environmental factors such as freeze-thaw, wet-dry, etc.

6. Facilitate maintenance operations such as correction of surface and line and ballast cleaning operations.

7. Resist fouling and "cementing".

8. Minimize climatic influences (frost heave, swelling, etc.,) in the roadbed.

<u>Ballast Failure</u>

In general ballast is considered to have "failed" or to be inadequate if the foregoing ballast functions are not provided. It should be emphasized however that many interrelated factors can contribute to "degradation" or loss in functional ability of a track support structure--ballast is only one component in the system.

In the following, a number of ballast material characteristics are discussed which are believed to be related to or contribute to a "<u>loss in functional ability of the track support system</u>".

1. Inadequate Ballast Thickness

   The ballast layer thickness may not be thick enough to properly reduce stresses imposed on the subgrade. This may lead to excessive elastic deformation and/or rutting in the subgrade and can contribute to many other types of distress.

2. Inadequate Ballast Resiliency

   Because of the dynamic action of imposed loading, especially the "unsprung" weight of locomotives, it is desirable to have a "resilient" support system. Major contributors to the resiliency are the ballast and roadbed. In some cases, "cementing" of the ballast has been reported which produces a layer of undesirable "rigidity". Experience has shown that limestone and certain types of slag are susceptible to "cementing."

3. Degradation of Ballast

   Normally, "open-graded" ballast is placed which facilitates maintenance operations and is "free draining". If the ballast breaks down or "degrades" because of climatic and/or loading factors, it may lose its "open graded" characteristics. Additionally, it some cases "cementing" may occur as described above.

4. Ballast Pumping

   When repeated loads are imposed on the ballast, stresses at the ballast-roadbed interface may be sufficient to initiate a condition whereby the roadbed and ballast start to intermix and cause ballast "fouling". As time progresses and as track ballasting and resurfacing operations continue, a substantial amount of ballast can be forced into the roadbed and/or the roadbed is forced into the ballast. When this occurs, a "ballast pocket" may form. Additional discussion of "ballast pockets" is presented in a subsequent section entitled: "Roadbed Failures". In general, the intermixing of ballast and roadbed materials is undesirable due to the adverse effects of the roadbed fines on the strength and free-draining properties of the ballast.

5. Ballast Permanent Deformation

   If the ballast material does not possess adequate stability, repeated loading will cause excessive permanent deformation in the ballast layer. This may contribute to a loss of surface of the rail system. In many cases, inadequate "densification" or compaction contributes to an accumulation of permanent deformation while in other cases, the ballast, even when properly compacted does not possess adequate stability for the specific conditions(loading, ballast thickness, subgrade support, etc).

6. Fouled Ballast

   Excessive fines content of ballast contributes to a loss of "free draining" nature, a decrease in shear strength and a loss in the ability to facilitate maintenance operations. The fines that contribute to "fouled" ballast can come from external sources, degradation of the ballast, and/or pumping roadbed.

SUBBALLAST MATERIALS

Functions of Subballast

According to Spang (2.21), the functions of the subballast layer also called protective blankets are:

1. Distribution of ballast pressure and traffic loads.
2. Damping of vibrations.
3. Filtering of the layers bordering above and below.
4. Protection of the subgrade or foundation layer against damage caused by trust.
5. Preventing rain water from penetrating into the subgrade.

Types of Materials Used as Subballast

The following materials are normally considered for use as subballast:

1. granular materials such as crushed stone, sand-gravel, slag, etc.
2. stabilized soil (soil-cement, lime-soil, etc.)
3. asphalt treated aggregate

Polyvinyl chloride (PVC) and woven or non-woven fabric have been used as a separation layer between the granular material and the subgrade. There are indications that certain of these materials may have potential as a separation layer, but very limited documented results are available. Additionally, the inclusion of PVC or a fabric may cause great difficulties during ballast cleaning operations.

Other Considerations

When granular materials are used as subballast, it is important to have a gradation which will satisfy filter requirements and frost susceptibility requirements (when frost line reaches subballast).

The required thickness of a subballast layer may vary but normally ranges from about 6 to 12 inches (15 to 30 cm) (2.1, 2.21). Minimum thickness requirements are normally controlled by construction equipment considerations.

ROADBED

Function of Roadbed

According to Hay (2.5) the roadbed is "The regular, prepared subgrade on which are laid the ballast section, ties, and rails." Hay (2.5) states that the roadbed (ther terms roadbed and subgrade will be used synonymous herein) has a three-fold function:

1. The weight of the tracks and ballast as well as the superimposed loadings are supported as uniformly as possible.
2. Drainage is facilitated (use of raised grade).
3. A smooth, regular surface is provided on which the ballast section and track structures can be laid to the established grade.

Types of Roadbed Failures

Roadbed failures or inadequacies include:

1. Pumping of roadbed soil into the ballast.
2. Softening, rutting, and lateral movement of roadbed soil.
3. Frost heaving and moisture induced shrink-swell of roadbed soil.

A recent proposed revision to the AREA Manual (2.18), Chapter 1, Part 1 - Roadbed; Section 1.4 - Maintenance, contains the following discussion of these aforementioned failures or inadequacies.

a. Subgrade Pumping

Saturated subgrade soils may be pumped up into the voids of ballast by repetitive traffic loads. Fine sand, silts and clays are susceptible to pumping when water is present. In new construction and in major track upgrading to carry increasing traffic, such pumping action is reduce by using a layer of compacted subballast material graded to act as a filter.

In existing track, pumping occurs most frequently under poorly maintained rail joints. Muddy ballast at a rail joint should not, however, be assumed as a sign of subgrade pumping. Such fouling is sometimes caused by internal abrasion of the ballast pieces under heavy dynamic loads. On main lines in particular, old gravel and ballast underlying more recently placed crushed ballast material seems to form an effective filter against upward pumping of the subgrade. Cause of ballast fouling should be checked by excavation into the ballast section before remedial action is chosen.

Subgrade pumping can only be eliminated by holding the unstable soil in place. It is indicated that this can be accomplished by a) removing fouled ballast and reconstructing the track using a subballast filter layer, b) laying a membrane under the ballast, or c) applying a bituminous spray to the top of the roadbed.

It is also stated that the application of more top ballast without removal of the fouled ballast is not recommended, as pumping will foul the new ballast.

b. Softening and Squeezing of Roadbed

Areas where track settles repeatedly under traffic, requiring periodic surfacing, are sometimes called "soft spots". The settlement of track will be accompanied by soil squeezing up between the ties or out at the track shoulders, or bulging on the roadbed slopes.

Soft spots usually occur where there are plastic subgrade soils, water trapped in the roadbed, and heavy traffic. Soft spots usually develop as follows:

1) When track is laid on plastic subgrade soil which is not well compacted or stabilized, traffic loads will cause a depression to form under each tie, capable of holding free water.

2) As the water cannot evaporate, it remains in these depressions, and softens and weakens the plastic soil.

3) Overstressed by traffic loads under the ties, the soil squeezes up to a position of lower stress between ties, or where there is sufficient thickness of ballast to prevent this, moves laterally to the track shoulder.

4) In either case a ridge of impermeable soil is raised around each depression or ballast pocket, which is thus capable of holding a large amount of water. Clay at the base of the ballast pockets continues to soften and squeezes out of position, making the condition self-perpetuating.

If track can be taken out of service temporarily, and the cost is warranted, soft spots and ballast pockets can be cured by:

1) Excavation and replacement of the plastic subgrade soil with more stable soil.
2) Stabilization of the subgrade soil.
3) Introduction of a surface cap of stone screenings or similarly graded material on the top of subgrade to reduce the amount of rainfall reaching the subgrade.
4) Introduction of a waterproof membrane at the surface of subgrade to prevent it from becoming saturated.

If track cannot be taken out of service, soft spots and ballast pockets can be improved by one of the following:

1) Drainage of the subgrade if water can be drained to a level lower than the base of the ballast pockets. The method of drainage should be based on a field investigation. Drainage pipes usually silt up. Excavation of track shoulders and replacement with pervious materials graded to act as a filter may offer relief.
2) Stabilizing the subgrade soil with cement grout.
3) Stabilizing the subgrade by the controlled addition of lime if conditions and subgrade soils are suitable.

c. Frost Heaving

Frost heaving of roadbed and ballast is caused by the simultaneous presence of fine-grained material, water, and freezing temperatures. Moist soils display volume change upon freezing, but significant volume increases occur only when ice lenses occur or develop. Rough track is caused when difference in volume change of subgrade soils develop over short distances along or across the track.

These differences may be <u>more or less</u> than the average heaving of the track. More heaving than average (rise in track) occurs typically at farm and road crossing; less heaving than average (dip in track) occurs typically at road crossings where salt is applied. A change from a heaving condition may occur at bridge approaches or at the end of rock cuts. A large proportion of all types of heaving occurs in regular track where no unusual features are present but a change in subgrade soil occurs.

Rough track, caused by frost heaving, causes excessive wear on both track and rolling stock and involves the danger of accidents, unless slow orders are applied or track conditions improved. The most common method of improving the track surface is by the temporary shimming of ties. However, this is expensive, reduces the service life of wood ties, and requires an experienced labor force to be kept on hand, particularly at the start of freezing and thawing seasons when heaving and subsidence occur relatively suddenly. Concrete ties are normally not shimmed more than a small amount. Canadian National Railways personnel (2.6) have indicated the use of salt for reducing frost heave of track structures.

LOADING ENVIRONMENT OF THE TRACK SYSTEM

The loading environment for the track system must be though of in a three-dimensional system because loads are imposed in the vertical, longitudinal and lateral directions. In the vertical direction, the main source of loading is the wheel loads of the train. In the longitudinal direction, the loading can be cause by braking or traction of the train and thermal expansion-contraction of the rails when continuously welded rails are used. Thermal forces are believed to be maximum toward the ends of the CWR.. In the lateral direction, loading can be cause by motion of the wheels (on curves, braking, etc.,) and also by thermal expansion and contraction.

The main emphasis in the Ballast and Foundation Materials Research Program is the vertical loading aspect although some attention is being given to the lateral restraint capability effected by various types of ballast and ballast section geometry.

Vertical Loading Environment

In considering the design of the track support system, a number of aspects of the vertical loading are important including:

1. Magnitude of gross vertical loads, axle loads, and wheel loads.
2. Dynamic loading considerations.

3. Number and frequency of load applications

a. Magnitude of Gross Vehicle Loads, Axle Loads, and Wheel Load

Gross vehicle loads range upward to 160 tons (145 metric tons) for some cars to 190 or 210 tons (175 to 195 metric tons) for locomotives. Table 2.8 summarizes pertinent facts concerning gross load for various cars and locomotives.

Axle loads obviously depend on the number of axles per car or per locomotive, the distribution of load, and the gross load. Typically, cars have either 4 (most common) or 6 axles, thus the load per wheel would range from about 30,000 to 40,000 lb (13,600 to 18,100 kg) for typical heavy, loaded cars. Maximum axle loads for locomotives are 65,000 lb (29,500 kg) or more.

Based on the foregoing, it is obvioius that the upper limit on static wheel loads is on the order of 25,000 to 40,000 lb (11,300 to 18,100 kg).

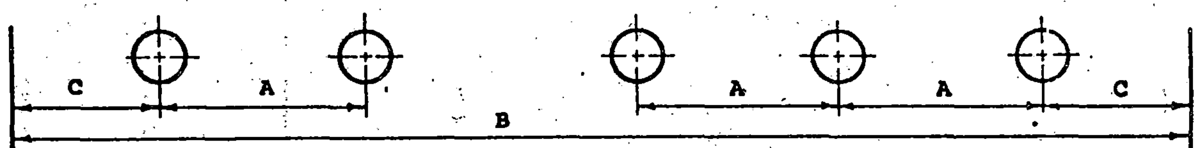
b. Dynamic Loading Considerations

Dynamic or impact loading caused by a train in motion can cause magnitudes of loading substantially greater than the aforementioned "static" values. The dynamic effects have not been well defined but Meacham, et al. (2.7) indicate that impact loading is twice the static loads. The railroad industry has generally accepted a 50 percent increase, and Talbot (2.8) suggested a one percent increase for each 1 mph (1.6km/h) speed increase over 5 mph (8.0 km/h). For concrete tie design, dynamic loading is normally assumed to be two and one-half times the static loading.

c. Number and Frequency of Load Applications

Main track traffic volumes range from one million or less gross tons (0.9 million or less gross metric tons) per year to 50 to 60 million gross tons (45 to 54 million gross metric tons) annually. If one assumes that an average car weighs 80 tons (73 metric tons), 50 tons (45 metric tons) of pay load and 30 tons (27 metric tons) empty, and one-third of the annual tonnage arises from empty car movement, then Figure 2.1 depicts the average car movements per year as a function of gross track tonnage. For example, if the gross tonnage is 30 million tons (27 million metric tons), the the number of car movement per year is about 475,000. If it is further assumed that each car has four axles, then the number of load applications is about 1.9 million per year.

Table 2.8. Typical Rolling Stock Data (From Ref. 2.13).



Rolling Stock	Maximum Load		Axle Centers "A"	Coupled Length "B"	End Axle to Coupling Line "C"	Wheel Diameter	Load per Inch of Wheel Diameter
	Per Wheel	Per Car					
<u>4-Wheel Trucks</u>							
Ore car H5	24,338 lb.	194,700 lb.	5'-0"	24'-0"	1'-11"	33"	738 lb.
Covered Hopper Car G75	32,875	263,000	5'-10"	54'-6 1/2"	3'-3 1/4"	36"	913
Wood Chip Car J39	32,925	263,400	5'-10"	65'-0"	3'-9 1/2"	36"	915
U28B Locomotive	33,750	270,000	9'-4"	60'-2"	7'-4"	40"	844
Monsanto Tank Car	39,375	315,000	6'-0"	67'-2 1/2"	3'-9 11/16"	38"	1036
<u>6-Wheel Trucks</u>							
AAR Car No. 1	26,250	315,000	4'-6"	55'-0"	3'-9"	30"	875
AAR Car No. 2	30,000	360,000	5'-0"	60'-0"	4'-6"	33"	909
AAR Car No. 3	32,875	394,500	5'-6"	65'-9"	5'-0"	36"	913
SD-45 Locomotive[(a)]	35,000	420,000	6'-9 1/2"	65'-8"	6'-1/2"	40"	771

(a) Up-dated as per <u>Car and Locomotive Cyclopedia</u>, 1974

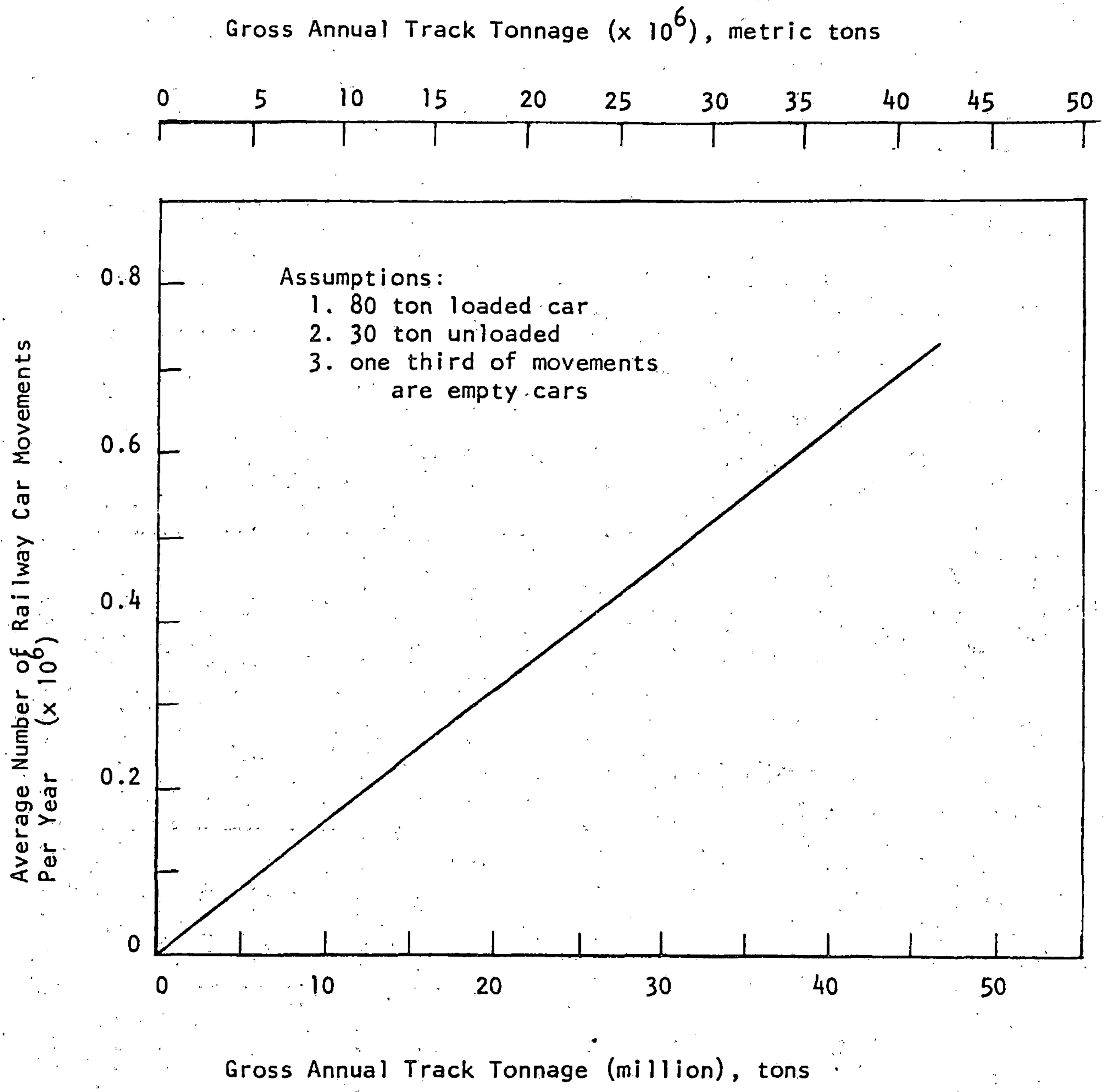


Figure 2.1. Average Number of Railway Car Movements Versus Gross Annual Track Tonnage.

The frequency of load applications imposed on a track is not constant as a train passes because the distance between the trucks of adjacent cars does not normally equal the truck spacing for a particular car.

In order to look at the frequency of load applications at a point, one must know the axle spacing, truck spacing, and spacing between trucks on adjacent cars.

For a typical car configuration, Figure 2.2 depicts the elapsed time between load applications as a function of speed for adjacent axles on the same truck and for the centerline of adjacent trucks on the same car. For example, the elapsed time between load applications of adjacent axles on the same truck is about 0.07 sec. for a speed of 60 mph but is about 0.4 sec. for a speed of 10 mph. Table 2.9 summarizes the FRA recommended maximum permissible train speeds for various classes of track.

Lateral Loading

Lateral loading of the track system can occur due to a) lateral wheel loads on tangent sections, b) thermal expansion and contraction, c) the "runin" and "runout" effect and steady draft and buff forces, and/or d) the jack-knifing of cars or locomotives due to severe braking or curve negotiation.

Lateral wheel loading characteristics apparently are not well defined. Studies by Meacham, et al (2.7) indicate that lateral wheel loads normally do not exceed 40 percent of the vertical loads but at times may increase to 60 percent of the vertical load. For design purposes, a lateral load of 90 percent of the vertical is suggested by Meacham, et al (2.7). Tests made with in-service cars have indicated lateral wheel-rail forces ranging from 16,000 to 21, 000 lb (7,300 to 9,500 kg) (2.9) up to 31,000 lb (2.17).

Lateral forces due to "sunkinking" are not well defined although Magee (2.2) suggests a 300 lb (136 kg) lateral restraint per tie in an unloaded condition.

ENVIRONMENTAL CONSIDERATIONS

Environmental exposure conditions have a very substantial effect upon not only the instantaneous behavior of the railway support system, but also on its long term behavior and performance. The very nature of the component materials and the track system geometry make them extremely sensitive to certain exposure conditions.

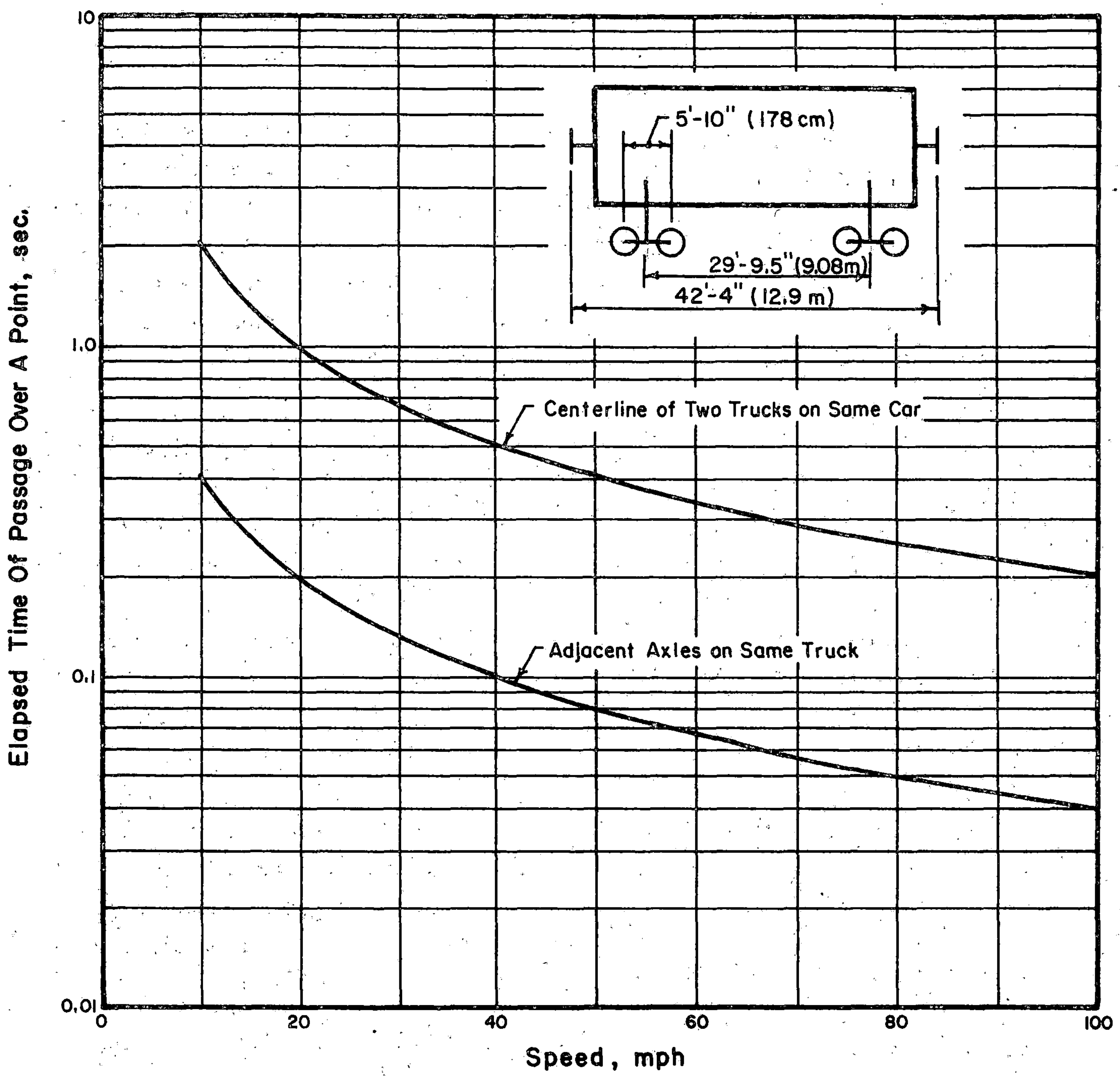


Figure 2.2. Elapsed Time of Passage Versus Speed for Typical Car.

Table 2.9. FRA Allowable Operating Speeds
(From Ref. 2.15)

Over track that meets all of the requirements for	The maximum allowable operating speed			
	For Freight Trains		For Passenger Trains	
	mph	km/h	mph	km/h
Class 1 track	10	16	15	24
Class 2 track	25	40	30	48
Class 3 track	40	64	60	96
Class 4 track	60	96	80	128
Class 5 track	80	128	90	145
Class 6 track	110	177	110	177

The environmental factors of apparent significance are temperature and moisture. Certain aspects of temperature conditions such as freeze-thaw and frost heave are important not only in terms of design of the support system but also in terms of material quality and property evaluation.

The moisture regime that exists in the support system is also an important consideration. Factors such as precipitation, water table position, permeability of ballast and foundation materials, local drainage conditions, etc., all have an influence on the moisture regime of the system.

For design and analysis considerations, it is desirable to quantify the temperature and moisture regimes both as a function of space and time. The present state of technology however, does not allow for precise quantification. In some cases however, semi-empirical approaches are available.

Temperature

The temperature regime that will exist in a typical railway support system as a function of time and space is an important consideration.

The Corps of Engineers (2.10) has used data from 361 U. S. Weather Bureau Stations to develop Air Freezing Indices for the U. S., Figure 2.3. The Air Freezing Indices do not define the temperature regime as a function of space or depth, however. Other data are available concerning maximum depth of frost penetration, daily air temperatures, etc., which are important, but cannot be used to accurately define the temperature regime in the railway support system. In 1970, Dempsey and Thompson (2.11) developed a rational computer-based procedure that utilizes fundamental thermal properties of the materials and pertinent climatic data to predict the temperature regime as a function of both time and space. Experience with this procedure has shown that it is quite adequate for predicting the temperature regime for both time and space in a pavement system. Even though this procedure was developed primarily for use with pavement structures, it can be used for characterizing the temperature regime in the railway support system.

As an illustration of the type of data that might be developed with the Dempsey and Thompson procedure, Figure 2.4 illustrates the freeze-thaw cycles occurring at various depths in a typical highway pavement. Also, Figure 2.5 illustrates the frost line under a typical pavement as a function of time.

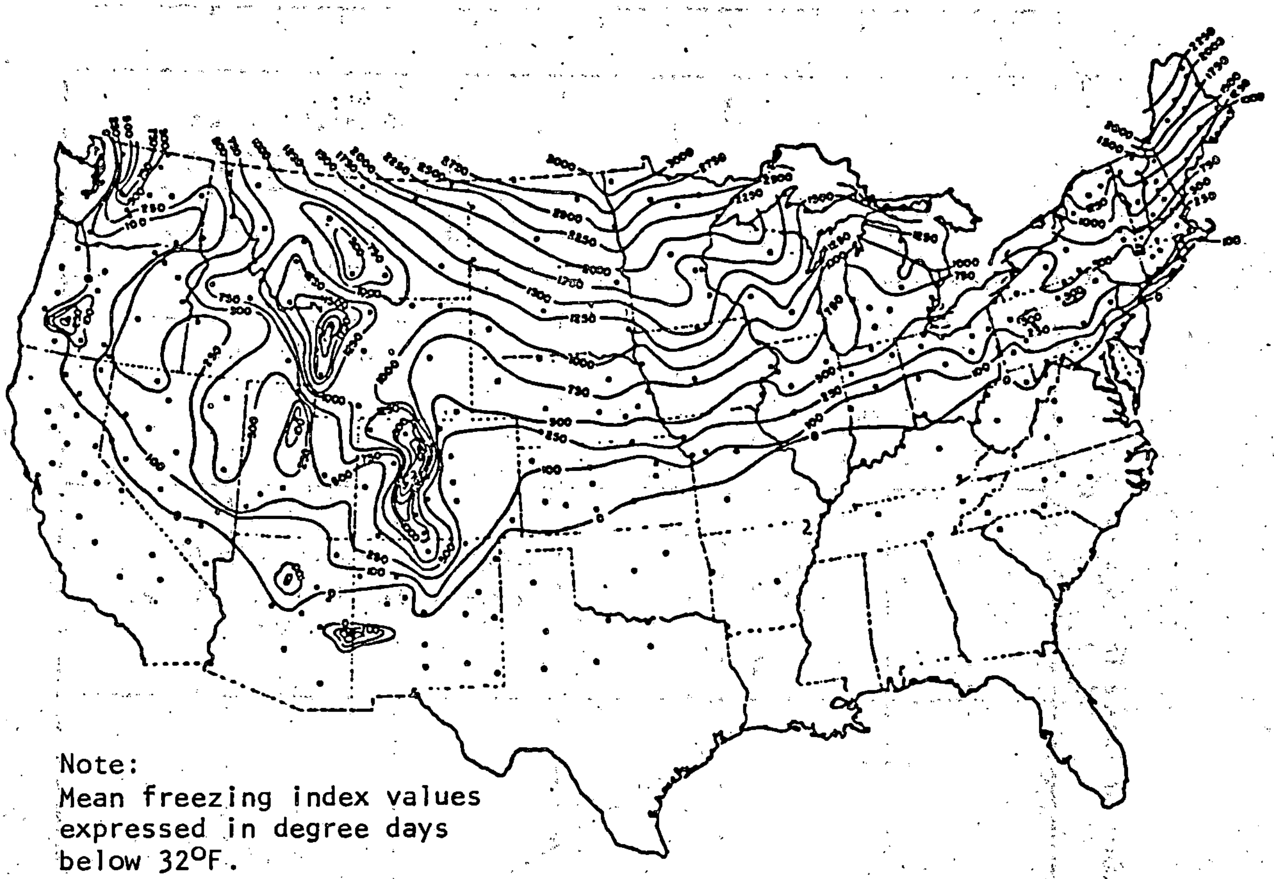


Figure 2.3. Distribution of Mean Freezing-index Values in Continental United States (From Ref. 2.10).

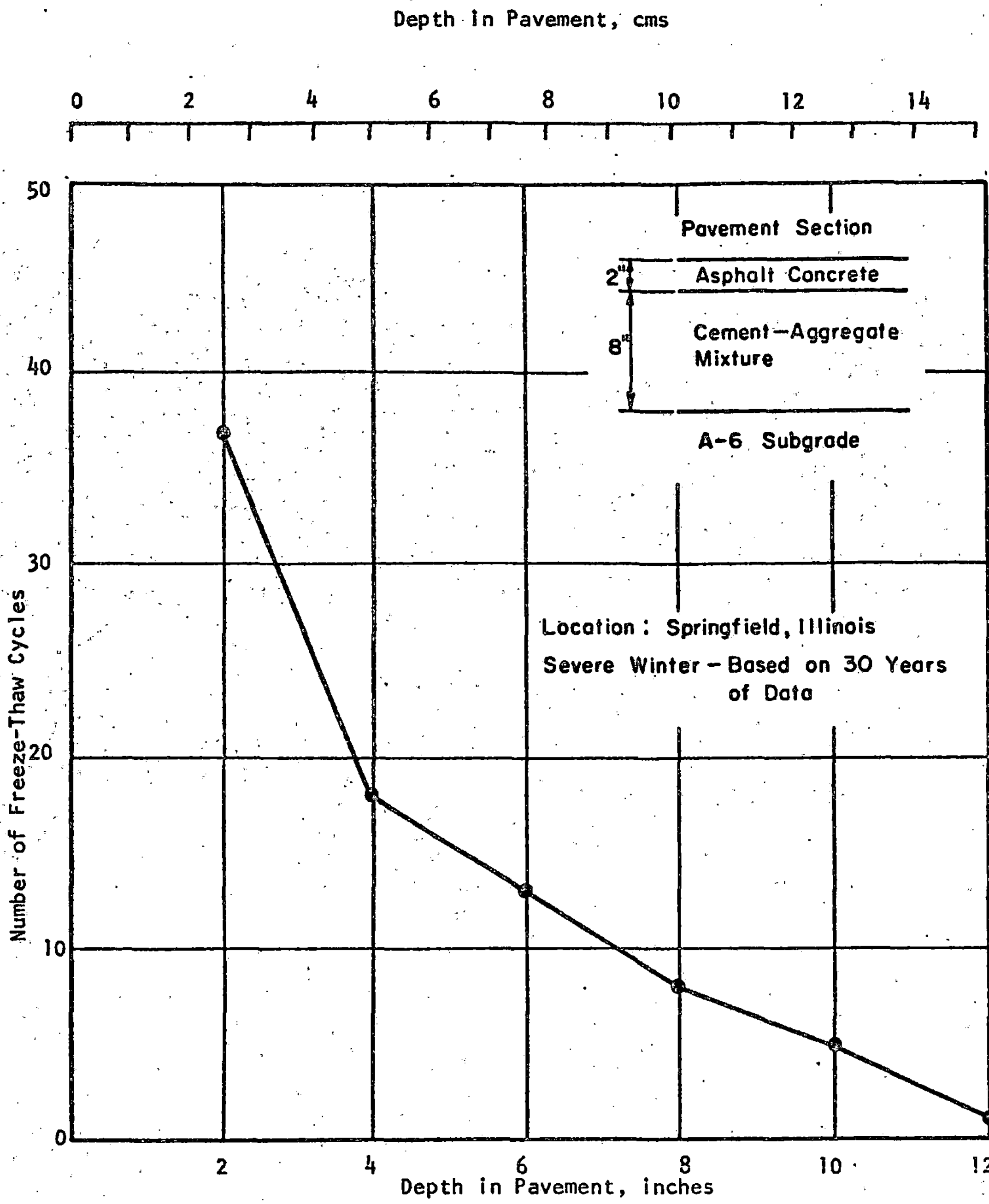


Figure 2.4. Number of Freeze-Thaw Cycles at Various Depths in the Pavement Shown Based on Thirty Years of Climatic Data at Springfield, Illinois (From Ref. 2.16).

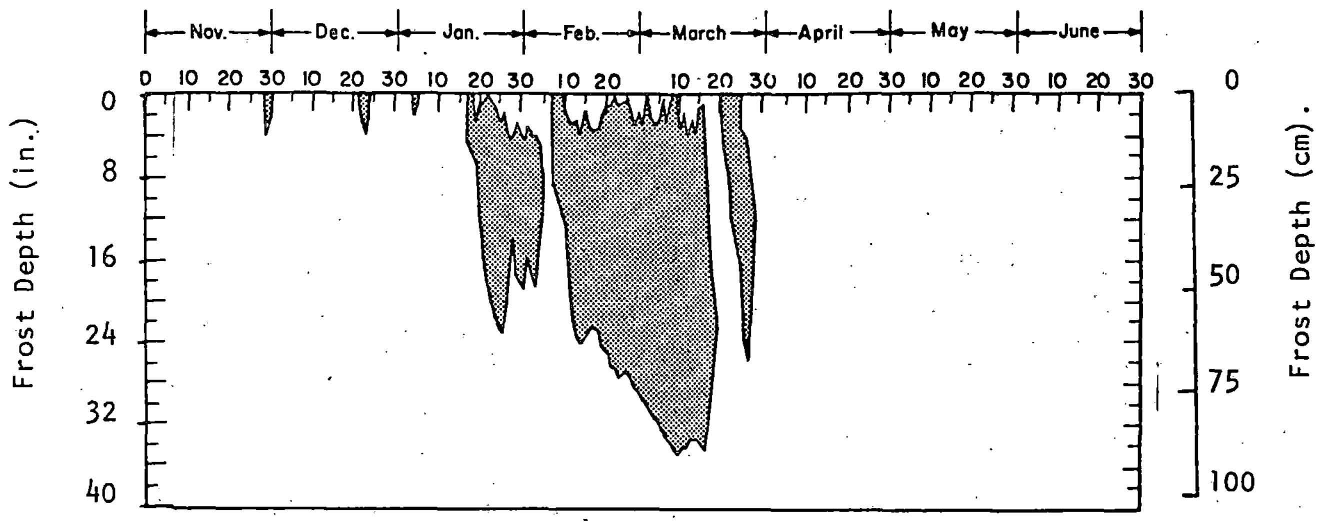


Figure 2.5. Frost-Line Position in a Pavement Test Section at 1400 hours (From Ref. 2.11).

Moisture

Current technology has not advanced to the point that accurate prediction of the moisture regime in a pavement or railway system can be made as a function of time and space for non-isothermal, unsaturated conditions. It does not appear that semi-empirical procedures are available for moisture regime prediction either.

It is a well known fact however, that the moisture regime in a pavement system (and thus probably also in a railway support system--especially the foundation soil layer) varies as a function of time and space.

ECONOMIC CONSIDERATIONS

At present, ballast selection criteria are largely based on initial cost, which includesaspects such as availability and transportation costs and neglectsfor the most part, service life and performance level considerations, except as such considerations are subjectively incorporated as a result of experience. This situation provides an improper perspective of the importance of careful ballast selection, since the exclusion of the latter two long-term, in-service cost factors prevents proper assignment to "true ballast cost". The influences of ballast on other material and labor accounts must be considered. Of the cost factors affected by ballast properties (i.e., rail, ties, other track materials, and track laying and surfacing) track surfacing is the most significant and it represents nearly one-quarter of all Maintenance of Way and Structures expenditures. This source of expenditure reflects the frequency with which track smoothing, surfacing, and reballasting operations must be performed. Unfortuantely, the relationship between these costs and the degree of stability of various ballasts has not received quantitative analysis.

By inspecting the present nature of ballast analysis and thickness design, it is obvious that little progress can be expected if these matters are left purely to the discretion of individual railroads. Typically, each railroad purchases ballast from on-line producers, loads its own cars, and hauls the ballast as far as necessary or practical. Seldom are ballast type cost factors kept; rather, ballast costs are lumped into system cost figures. The current system of ballast purchase and use fosters a certain resistance to change in ballast selection. Additionally, exclusive use of the products of on-line producers unnecessarily limits experience to just a few materials, thereby eliminating the ability to optimize ballast selection.

CHAPTER 3

BALLAST MATERIAL PROPERTIES

GENERAL

For efficient and effective utilization of ballast* materials, it is essential the engineer have a knowledge and an understanding of the properties and behavior of the materials. Because ballast may be subjected to large numbers of loadings and exposed to extreme weathering conditions, many types of procedures and techniques have been developed to evaluate ballast properties.

Required ballast properties are related to the functions of the ballast and subballast system (see Chapter 2) and to the loading and exposure conditions.

Two extensive annotated bibliographies on soil-aggregate mixtures and on ballast have been compiled and are included as additional references following Chapter 3 list of references.

EVALUATING BALLAST MATERIAL PROPERTIES

General

Tests for evaluating ballast materials basically fall into two categories:

1. tests for evaluating quality and
2. tests for evaluating load response or behavior

Many of the quality tests have been developed as indirect indicators of potential in-service behavior. These quality tests quite often evaluate characteristics of the individual particles making up the ballast mass. On the other hand, the tests used for evaluating ballast material load response typically are concerned with the load response of the ballast mass as opposed to the response of individual particles.

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* Throughout this report, subballast is considered under the broader term, ballast.

Testing Procedures Used to Determine Quality

a. Freeze-Thaw Tests

Several versions of freeze-thaw tests have been used to evaluate aggregate resistance to disintegration by freeze-thaw. AASHTO T 103 covers the essentials of the typical testing procedure. In general, either partial or complete immersion can be used during F-T, and the fluid can be either water or a water-alcohol solution. Although the AASHTO procedure does not recommend the number of F-T cycles, some authorities (AASHTO T 103) have recommended the use of as many as 50 F-T cycles. Resistance is measured by the change in gradation caused by the F-T testing. Rapid freeze-thaw tests are open to criticism in that they do not simulate the actual rates of temperature change to which the ballast is subjected. Certainly the accelerated tests using water-alcohol do not accurately simulate field conditions.

Particles with fractured surfaces may offer more opportunity for penetration of water and hence for freeze-thaw degradation. Also elongated, angular particles offer more surface area than do smooth, spherical particles.

Peckover (3.1) and West, et al. (3.2) feel the freeze-thaw test yields results useful for predicting resistance to field weathering.

b. Soundness Tests

Soundness tests such as ASTM C 88 and AASHTO T 104 use saturated solutions of sodium or magnesium sulfate to determine resistance of aggregates to disintegration. The test consists of five cycles of alternately soaking the aggregate in the saturated solution and then drying. Soundness is evaluated by calculating the loss or change in gradation as a percentage of the initial weight.

The soundness test relates to disruption of the rock by the growth of chemical crystals rather than by the growth of ice crystals during freezing and thawing and has been criticized because it does not simulate actual conditions. Additionally the soundness test has been criticized because it is not reproducible in different laboratories, especially if sodium sulfate is used. West, et al. (3.2) concluded the sodium sulfate test did not prove useful or reliable. Bloem (3.3) found that magnesium sulfate produced more consistent results than did sodium sulfate, possibly because temperature variations affect the magnesium sulfate test less.

Freeze-thaw tests apparently give results which predict the weathering resistance of aggregate better than does the soundness test (3.1, 3.2, 3.4). However, Raymond, et al. (3.5) found good correlation between field performance and the sodium sulfate soundness and freeze-thaw (AASHTO T 103) tests.

c. Absorption Test (ASTM C 127)

The absorption test is used to determine the bulk and apparent specific gravities of the aggregate along with the amount of water the aggregate absorbs. The absorption capacity of the aggregate is believed to be related to the weathering potential of the aggregate. However, based on the literature, there appears to be some controversy as to the significance of the test results. Peckover (3.1) feels the absorption test offers little in addition to the soundness test and thinks a direct measure of the porosity of the rock is more important.

Dalton (3.4) found high testing variations (as much as 60 percent of the acceptable limit of 0.5 percent) for the absorption test. Dalton (3.4) also concluded the absorption limit may allow approval of ballast which may break down too rapidly due to field weathering. Raymond, et al. (3.5) found good correlation between the freeze-thaw, soundness, and absorption test results.

d. Durability Test

The California Highway Department has developed a test (California Test No. 229) to measure durability by agitating aggregate in water and measuring the change in gradation. Recently, this procedure has been included in AASHTO Standards (3.6) as AASHTO T 210.

e. Los Angeles Abrasion Test

The Los Angeles Abrasion test (ASTM C 131 and C 535 and AASHTO T 96) consists of rotating a steel drum containing aggregate and steel spheres. The change in gradation as measured by the loss in percentage retained on certain sieves is expressed as the Los Angeles number.

AREA specifications recommend that LA Abrasion losses be limited to a maximum of 40 percent (3.7). High quality aggregate base course materials considered acceptable for highways and airfields normally have losses less than 40 to 50 percent.

Attempts have been made to correlate Los Angeles Abrasion data with field degradation. West, et al. (3.2) obtained good results by running both wet and dry tests. Scalzo (3.8) found the values correlated well except for basalt. According to Goldbeck (3.9) the L. A. Abrasion test "gives a pretty fair indication of the service value of ballast."

Dalton (3.4) found that the L. A. Abrasion test used alone was not an entirely reliable indicator of ballast quality. Dalton indicated that ballast with abrasion losses slightly in excess of the maximum allowable may be satisfactory if the resistance to freeze-thaw is good.

f. Flakiness Index

A procedure for determining the content of "flaky" or elongated particles has been developed and is covered by British Standard 812 (3.10). A flaky particle is defined as one whose least dimension is less than 60 percent of its mean size.

The Canadian National Railways specifies ballast "shall be free from an excess of thin or elongated pieces" and indicates the amount shall be no greater than 30 percent (3.1). A great variation (25 percent of mean) was found among tests of in-service ballast (3.1). Rad (3.11) found "flaky" particles tended to breakdown more rapidly in an impact test than did rounded particles.

g. Soft Particles; Clay Lumps and Friable Particles

Soft particles, clay lumps and friable particles are undesirable in a ballast material. ASTM C 235 has been recommended by AREA for evaluating soft particles. Basically, the test involves scratching ballast particles with a sharp brass rod. Particles are "soft" if a groove is made without the deposition of brass. AREA recommends that soft particles be limited to less than 5 percent (3.7).

Clay lumps and friable particles are limited to 0.5 percent by the AREA specification (3.7). ASTM C 142 describes clay lumps and friable particles as particles that can be broken with the fingers after the aggregate has been soaked in water for 24 $\pm$ 4 hours (3.12).

h. Shape, Angularity, and Surface Texture

Although it is possible to measure separately the shape, the angularity, and the surface texture of individual particles, the effects of these on ballast stability seem to be interrelated, and commonly a single number representing the shape, angularity, and surface texture of the aggregate is used. Huang (3.13, 3.14) developed a Particle Index test in which single sized aggregate is rodded in a rhombohedral mold and the void ratio is compared with the void ratio of uniform spheres. Recently Huang (3.15) modified the Particle Index test to use a standard CBR mold; the test has been adopted as a tentative ASTM standard.

Ishai and Tons (3.16) developed a pouring test for expressing geometric irregularity of particles by parameters which are related to volume, specific gravity, and aggregate porosity. The pouring test is conducted by allowing aggregate to flow through an orifice into a container to overflowing, leveling the aggregate, and weighing it. The packing specific gravity G_p, is determined by comparing the weight to the weight of glass beads which fill the container. The total geometric irregularity of the particles is expressed by the specific rugosity (S_{rv}):

$$S_{rv} = 100\left(1 - \frac{G_p}{G_{ap}}\right) \qquad (3.1)$$

where

G_p, G_{ap} = packing and apparent specific gravities (ASTM E 12).

The specific rugosity of smooth, uniform spheres is zero. Typical values for specific rugosity for some types of aggregate are:

Beach pebbles	2.45
Natural gravel	8.49
Slag	22.78

(One-sized particles, 1/2 in.-5/8 in.) (13 mm-16 mm) (From Ref. 3.16).

i. Petrographic Analysis

Petrographic analysis (ASTM C 295) is the microscopic examination of rock samples and the identification of the constituents and of any other properties the petrographer thinks significant.

Several investigators (3.1, 3.2, 3.17, 3.18) have attempted to correlate the results of petrographic analysis of the particles with field degradation. Unfortunately, results differ greatly among petrographers because of the subjectivity of the analysis. The CNR commonly uses petrographic analysis for ballast characterization (3.1).

j. Gradation

The particle size distribution is an important characteristic of ballast materials. Aspects such as maximum particle size, particle size distribution and proportion passing the number 200 sieve (0.074 mm) are determined. Grain size analysis can be determined by using ASTM C 136 or AASHTO T 27. The percentage passing the number 200 sieve can be found by using ASTM C 117 or AASHTO T 37.

k. Hardness

Several testing schemes have been developed for use in estimating the hardness of individual aggregate particles. The British use a test (British Standard 812) in which the aggregate is confined in a mold and subjected to 3170 psi (21.86 MN/m^2) pressure (3.8). The amount of breakdown or change in gradation is used to calculate the "Crushing Value".

The Germans report measuring hardness by means of a "toughness test". No description could be found of the details of the test.

Rad (3.11) proposed the use of an impact test which involves dropping a weight a specified distance onto several particles and measuring the change in size due to a given number of drops. Rad (3.11) found rounded particles to be less susceptible to impact than were sharp particles which were less susceptible than flat or elongated particles.

Scalzo (3.8) attempted to correlate field degradation of ballast materials with the crushing value. In general, he found that good performance of ballast was achieved if the "crushing value" was less than 20 percent.

l. Plasticity of Fines

Excessive fines content normally is not permitted for high quality ballast materials. Some lower quality ballast and many subballast materials may have a substantial amount of fine materials.

In general, it is found that stability of the granular material is adversely affected by fines with high plasticity. Atterberg limits (ASTM D 423 or AASHTO T 89 and T90) can be used to determine the plasticity characteristics of the faction finer than the number 40 sieve (0.425 mm).

It appears typical ballast specifications do not contain any limitations concerning plasticity characteristics. Subballast materials in general follow limits set by ASTM D 1241 for Soil-Aggregate Mixtures which require that the liquid limit be less than 25 and the plasticity index be less than 6.

m. Electrical Resistance

The electrical resistance characteristics of ballast material are important with respect to the track signal circuit. This resistance is influenced by a number of factors including conductivity of the ballast particles, "dirty" ballast, moisture, etc. Leakage of current from one rail of a circuit to the other rail is undesirable. Attention is given to the problem of electrical resistance of ballast in the A.A.R. manual on American Railway Signaling Principles and Practices (3.55). Under the most adverse conditions, the minimum allowable ballast resistance should not be less than 2 ohms per 1000 ft of track for circuits 4000 to 5000 ft in length (3.55). Details of a procedure for evaluating ballast electrical resistance and typical values of resistance for a number of ballast materials can be found in Reference 3.56.

Strength Tests

A variety of tests exists for determining the strength or resistance to deformation of granular materials. The tests particularly relevant to ballast materials evaluation are included in this section.

a. California Bearing Ratio (CBR), ASTM D 1883; AASHTO T 193)

The CBR testing method determines the "strength" of compacted laboratory specimens by comparing the load and penetration of the sample to that of a well graded, highly compacted crushed stone. The standard for high quality crushed stone is a CBR equal to 100. The laboratory testing is conducted according to ASTM D 1883 or AASHTO T 193. Additionally, in-situ strength of granular materials can be determined with the CBR test. In general, graded aggregate materials have compacted CBR values ranging from 40 to more than 100; CBR values for fine-grained soils are typically less than 10.

b. Static Triaxial Test

Triaxial tests may be used to measure the shear strength, the apparent cohesion (c) and angle of internal friction (ϕ), and deformation or static modulus of materials.

Several types and sizes of triaxial cells have been used for testing granular materials. Four to six in. (10 to 15 cm) diameter specimens are typical. There also is a triaxial cell at the University of California, Berkeley, capable of testing 36 in. (91 cm) diameter specimens (3.19).

The Texas Triaxial Test uses a specimen 6 in. (15 cm) in diameter and 8 in. (20 cm) high. Gray (3.20) used the Texas Triaxial method to determine the effects of several variables (maximum size, gradation, etc.,) on the strength of a variety of aggregates.

Typical static triaxial test results for a granular material are depicted in Figure 3.1.

c. Shear Box

Klugar (3.21) and Pike (3.22) have developed procedures for determining certain strength properties (shear strength, angle of shearing resistance) by using a shear box. Details of the equipment and testing procedure can be found in Reference (3.22).

d. Plate Test (AASHTO T 221, T 222; ASTM D 1195, D 1196)

Repetitive and nonrepetitive static plate load tests can be used to evaluate both subgrade and aggregate layer support.

The test is conducted using rigid steel plates ranging from 5 to 30 in. (13 to 76 cm) in diameter and applying load by hydraulic jacks. The plate deflection at various loads is measured, and the modulus of subgrade reaction, k, is calculated by:

$$k = \frac{p}{\Delta}$$

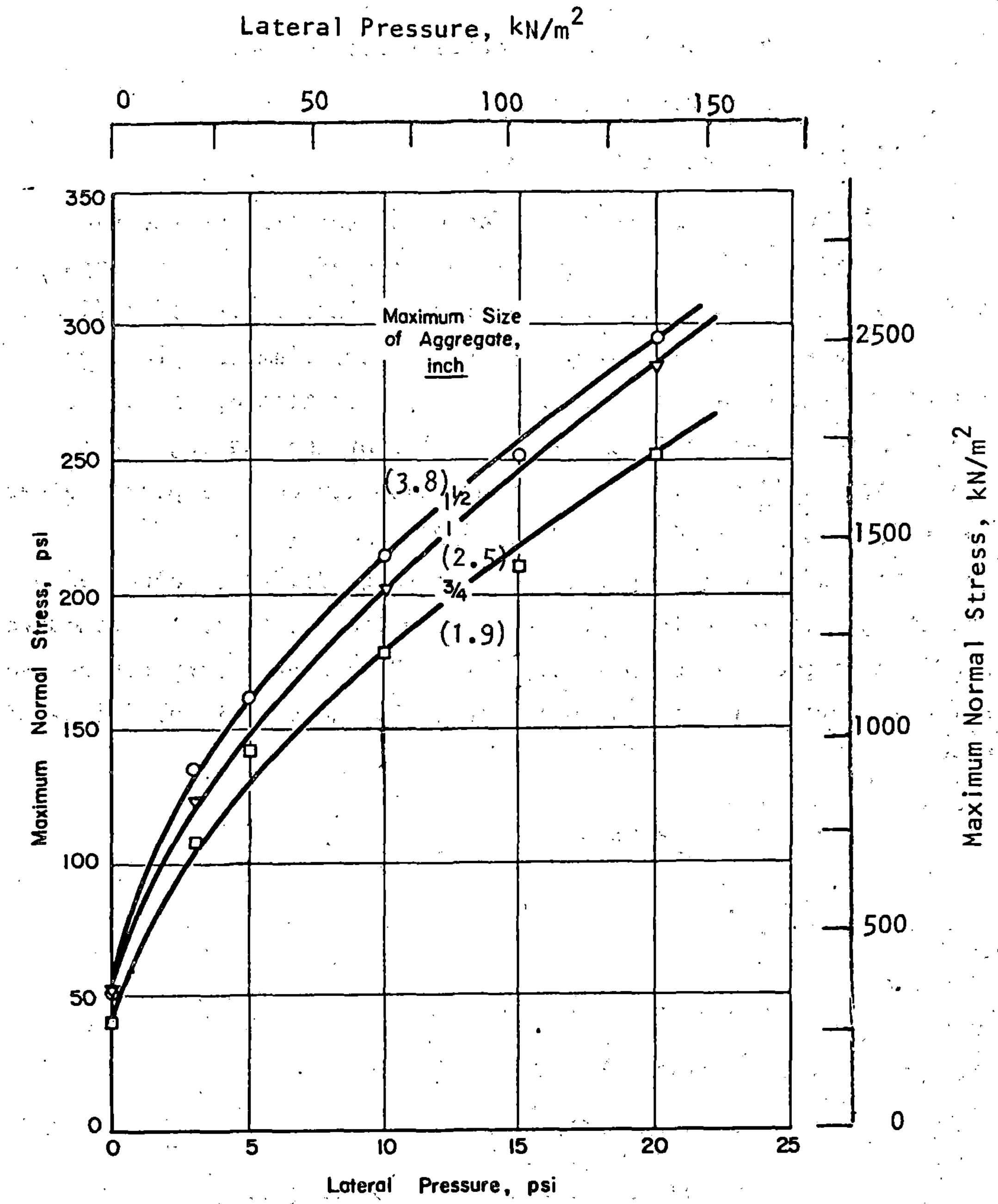

Figure 3.1. Relationship of Principal Stresses for Aggregates with Different Maximum Size, Gradation Index Equal to 1/2 (From Ref. 3.20).

where

k = modulus of subgrade reaction, pci

p = pressure, psi

Δ = deflection, in.

In general, the test results are affected by the size of the plate (the 30 in. (76 cm) diameter plate is most often used), and thus comparing the k-values obtained from tests using different plate sizes is difficult.

A type of plate load test using a 5 in. (13 cm) diameter plate has been developed by the Canadian National Railways for testing ballast in place. The "ballast bearing index" is the value of the loading plate pressure at 0.30 in. (7.6 mm) vertical plate displacement. Typical values of BBI range from 100 to 1000 psi (690 to 6900 kN/m^2) (3.23).

If a plate load test is conducted on the subgrade, an E-value can be calculated using Boussinesq theory. If a plate load test then is conducted on the overlying aggregate (or ballast) layer, Burmister theory can be used to determine an E-value for the granular layer by the use of the following equation:

$$\Delta = 1.18 \frac{pa}{E_2} F_2$$

where

Δ = plate deflection, in.

p = plate pressure, psi

a = plate radius, in.

E_2 = modulus of elasticity of subgrade, psi

F_2 = deflection factor which is a function of granular layer thickness and E value.

d. Burggraff Shear Device

The Burggraff shear procedure (3.25) is a field test that can be used to determine in-place stability but seldom is used today. In this procedure, a horizontal force is applied by a plate to an exposed vertical face until the material fails in shear. The method provides a convenient means of testing ballast type materials under in-service conditions. It is possible to adapt the device to laboratory use, but only field evaluations have been reported.

Huang and Lohmeier (3.25) found a definite correlation between Burggraff shear values and shear strength determined by triaxial testing.

Repeated Load Tests

Because granular materials are subjected to transient loading under typical in-service conditions it is desirable to evaluate the materials under similar conditions in the laboratory. Additionally, the repeated load response of granular materials is dependent on the stress state.

a. Repeated, Dynamic Load Tests

Several investigators (3.26, 3.27, 3.28, 3.29, 3.30, 3.31, 3.32) have used the conventional triaxial cell to measure the repeated load behavior of granular materials. Most often the confining pressure has been held constant and a repeated axial deviator stress has been applied, although Allen (3.31) and Brown and Hyde (3.30) compared constant and pulsating confining pressure effects.

1. Resilient Behavior

One extremely successful method of determining resilient behavior is through the use of triaxial testing equipment. A conventional triaxial cell is instrumented and vertical deflection measured at various magnitudes of confining pressure (typical range σ_3 = 3-25 psi, 20-170 kN/m^2) and various magnitudes of repeated axial stress (typical range σ_1 = 5-100 psi, 35-690 kN/m^2). The resilient modulus* for granular materials has been shown to increase as the cycled deviator stress increases and as the confining pressure is increased. A general expression for the resilient modulus is:

$$E_R = \overline{K}\theta^{\overline{n}} \tag{3.4}$$

where

E_R = resilient modulus, psi

$\overline{K}$ = a constant

θ = the first stress invariant ($\theta = \sigma_1 + 2\sigma_3$), psi

$\overline{n}$ = a constant which defines the slope of the line on a log-log plot

Typical results are shown in Figure 3.2.

Resilient modulus may be related directly to the confining pressure by an expression similar to Equation 3.4:

$$E_R = K\sigma_3^{n'} \tag{3.5}$$

Allen (3.31) and Brown and Hyde (3.30) tested granular materials using the above procedure by cycling the confining pressure as well as the deviator stress. It has been concluded the use of the constant confining pressure triaxial test is justified for characterization of the resilient response of granular materials (3.31).

The Hveem resiliometer has been used also to evaluate the resilient response of granular materials (3.33).

Raymond, et al. (3.5) and Wong (3.34) used one dimensional repeated load testing (oedometer) to determine the strain characteristics of ballast materials due to simulated field loading.

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*Resilient modulus is defined as the repeated deviator stress divided by the recoverable strain.

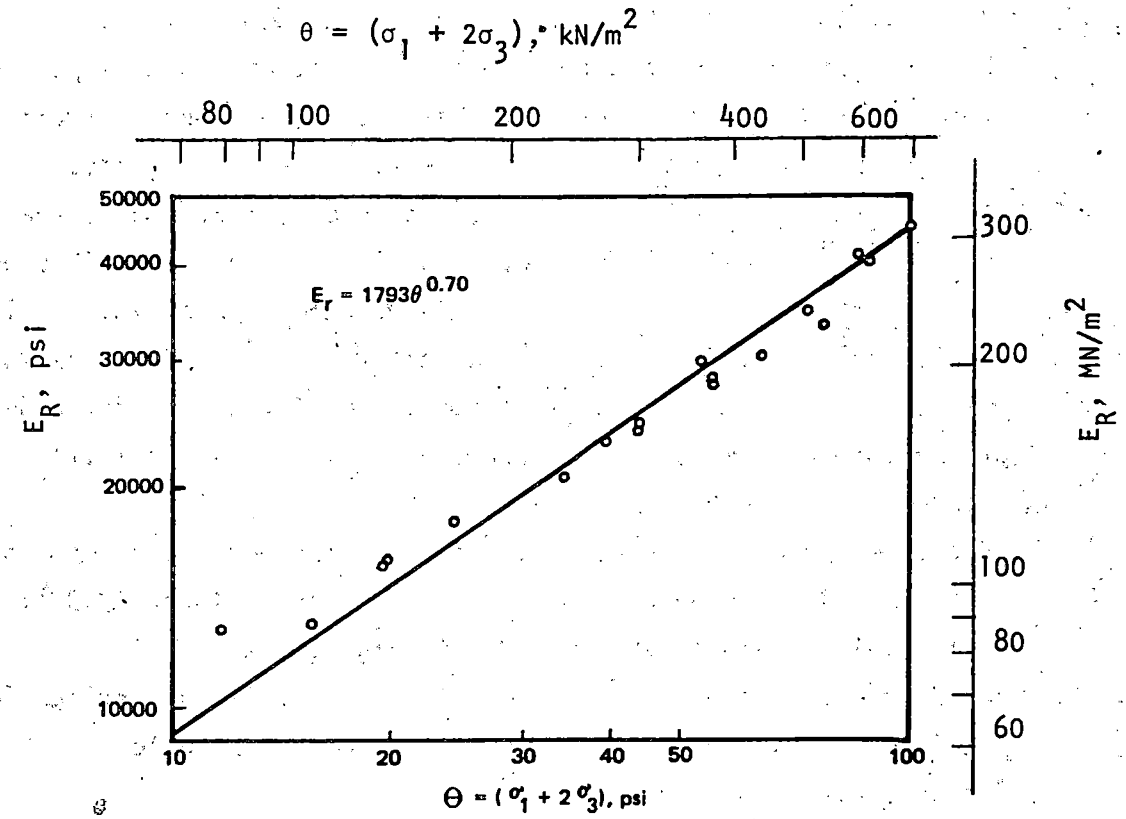

Figure 3.2. Resilient Modulus (E_R) as a Function of the Sum of the Principal Stresses, θ, Crushed Stone Specimen (From Ref. 3.31).

Tice (3.35) attempted to use a repeated load CBR test to determine the resilient behavior of a sandy silt but concluded the repeated CBR test was not a satisfactory test for measuring resilient modulus because the results were erratic.

2. Permanent Deformation

In addition to the determination of resilient properties of ballast materials subjected to repeated load, the permanent (also referred to as plastic or irrecoverable) deformation accumulated by the repeated application of a deviator stress is an important consideration. Barksdale (3.26) has shown the plastic strain to depend upon the number of load application, confining pressure, the aggregate type and gradation, the density, and the degree of saturation. Typical results are shown in Figure 3.3. Although the plastic deformation attributable to one cycle of loading is small, the accumulated deformation after a number of cycles may be appreciable. The rate of accumulation of cyclic plastic deformation has been shown to approach a limiting value fairly rapidly.

Two other important investigations measuring permanent deformation were done by Haynes and Yoder (3.36) and Holubec (3.37); some typical results are included in Figure 3.4.

Snyder (3.38) used a form of triaxial test utilizing springs for confinement to study the permanent deformation behavior of granular materials.

b. Complex Modulus

Another possible method for characterization of granular materials is through the use of complex modulus* as in the tests conducted by Coffman, et al. (3.39).

Field Tests

Numerous field tests have been reported in which ballast was evaluated under in-service conditions. But there are probably a large number of such

* Complex modulus is defined as the peak amplitude of sinusoidally applied stress divided by the peak amplitude of resulting strain (3.33).

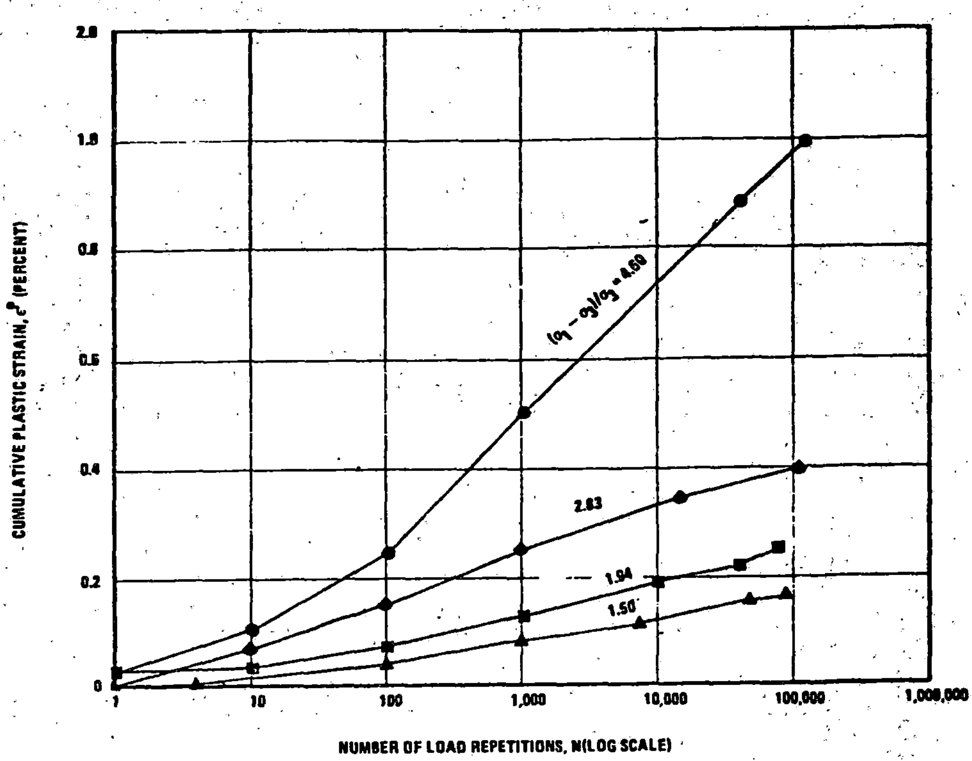

Figure 3.3. Influence of Number of Load Repetitions and Deviator Stress Ratio on Plastic Strain in a Porphyrite Granite Gneiss-Three Percent Fines, Confining Pressure Equal to 10 psi (69kN/m^2) (From Ref. 3.26).

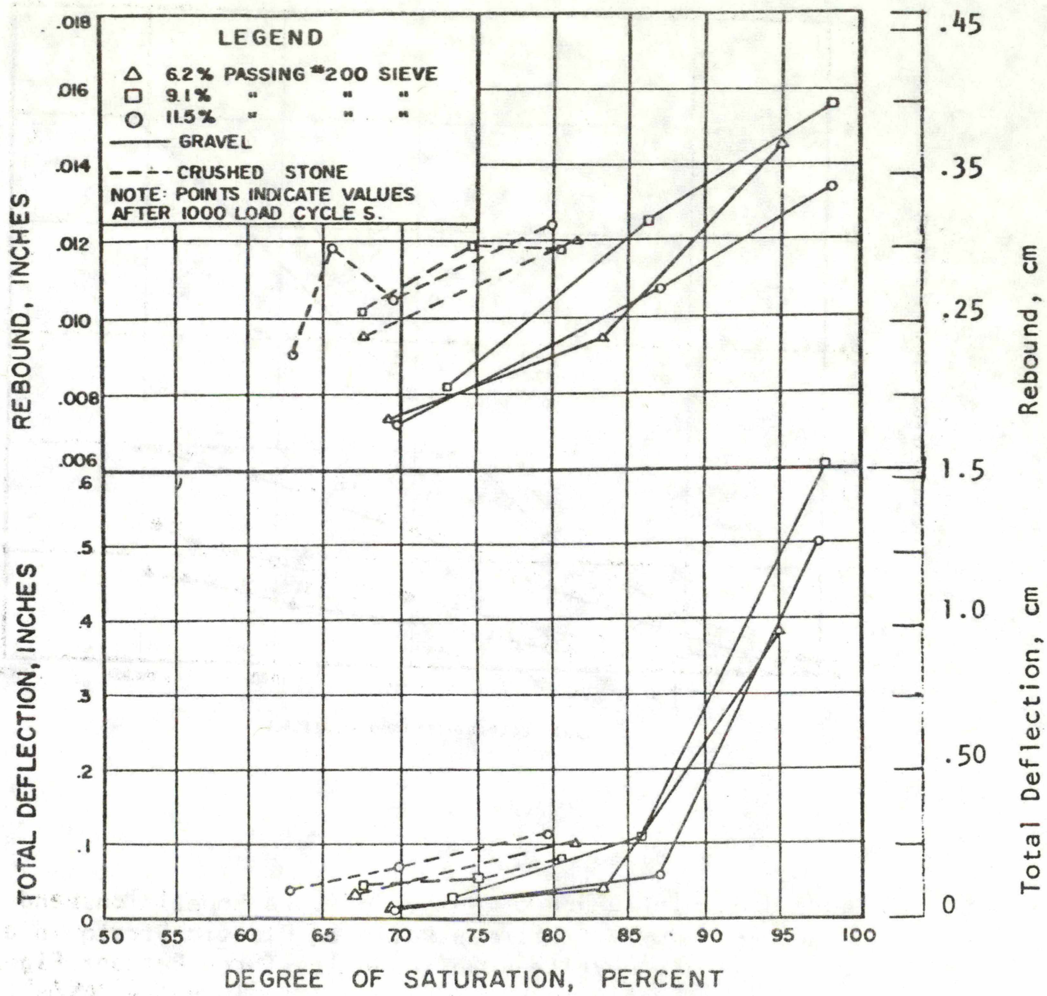

Figure 3.4. Deflection and Rebound After 1,000 Load Cycles (From Ref. 3.36).

field tests for which technical information either is not available or difficult to obtain.

Field testing can produce extremely valuable information as to ballast behavior if proper control is exercised and if behavior is properly measured. Numerous difficulties arise with respect to field tests and include:

1. difficulty in control of important parameters such as ballast and subgrade "strength" and ballast thickness,
2. the inability to isolate the effects of pertinent variables, for example, difficulty in isolating the effect of subgrade support and ballast "strength" properties,
3. the requirement that a long period of time is necessary in order to examine such aspects as "weathering", degradation, and long term loading effects, and
4. high cost involved when examining a large number of variables.

Regardless of the difficulties, field studies can provide valuable information that is extremely difficult or impossible to obtain under laboratory conditions.

As an example of the types of field tests that have been or are being conducted, the following summaries are presented:

a. Canadian National Railways

Field tests were conducted by the Canadian National Railways on ten in-service sections (3.4). The object of the test program was to compare the field durability of ten types of ballast with the CNR specifications and thus to determine the adequacy of the specifications. One side of each ballast section was underlain by a membrane which served to retain the fines generated due to loading and weathering. Findings to date indicate that CNR specifications appear to be adequate for prediction of in-service durability characteristics of certain ballast materials. However, based on the results, petrographic analysis was recommended as an additional test for some types of aggregate and for aggregate marginally passing the standard tests.

b. Kansas Test Track

One of the largest and most extensively monitored field tests currently underway is the Kansas Test Track near El Dorado, Kansas. The variables being evaluated do not relate directly to the ballast or roadbed (3.40).

c. ORE (Committee D 71)

A full scale test has been reported by ORE (3.41). The test incorporated different types of ballast in a support system and measured the vertical deflections of the rail due to traffic loading. The results correlated well with the results of the laboratory triaxial tests conducted on the same materials.

d. ORE (Committee D 117)

Another full scale field test recently has been reported by ORE (3.42). The report includes studies of the influence of ballast thickness and ballast compaction (density) on the settlement of in-service sections. In all cases the settlement for the three sections with 30 cm (12 in.) of ballast was greater than for the comparable sections with only 20 cm (8 in.) of ballast. The report also showed the consolidated track settled less than the non-consolidated track (3.42).

FACTORS AFFECTING TEST RESULTS

General

Variations in test results can be caused by numerous factors including inherent differences in material properties and apparent differences due to testing error. As a result, it is desirable to discuss some of the factors that affect differences in material properties.

Factors Affecting Quality Test Results

A number of factors may affect or cause variations in the results obtained from the quality tests that were discussed in a previous section of this chapter.

a. Freeze-Thaw Tests

Some of the factors influencing the results of the freeze-thaw tests are:

1) rate of freezing and thawing - Most of the present tests are conducted too rapidly to simulate the actual field

freezing and thawing conditions. Closer simulation of field conditions is desirable but is often impractical. The rapid freeze-thaw tests generally use a much greater temperature gradient for freeze-thaw cycles than would be representative of the field conditions. Especially open to criticism are tests which use low temperatures, e.g., 0°F for freezing, and high temperatures, e.g., 72°F for thawing. Most freeze-thaw cycles occur at temperatures varying only a few degrees from the freezing point.

2) <u>number of cycles</u> - The number of freeze-thaw cycles to which the aggregate is subjected should approximate the in-service value. Presently no standard exists for the number of cycles; it generally ranges from 16 to 50 cycles.

3) <u>presence of fractured surfaces and sharp corners</u> - Particles with fractured surfaces and sharp corners are more subject to water entry and hence freeze-thaw action than are well rounded gravels. Elongated, flat particles have more surface area per volume and present more imperfections for water entry than spherical particles.

4) <u>mineralogy</u> - The mineralogy of the aggregate may have a significant effect on the freeze-thaw characteristics of the aggregate because the mineralogy determines the size and continuity of voids and the inherent strength of the particles.

5 <u>adherence to procedures</u> - The repeatability of the freeze-thaw tests is not high and close adherence to standard test procedures is essential if testing error is to be minimized.

b. Soundness Test

Several factors, including the following, have been found to influence the results of soundness tests:

1) the type of solution (magnesium sulfate versus sodium sulfate) - The standard procedure permits the use of either type of solution, but Bloem (3.3) found magnesium sulfate consistently was more severe than sodium sulfate, especially in generating losses in the smaller size fractions.

The magnesium sulfate solution is believed to be less susceptible to temperature variations than sodium sulfate and hence the control of the test solution and the consistency of the results is improved by the use of magnesium sulfate (3.3).

Peckover (3.1) recommends only magnesium sulfate be used. However, Raymond, et al. (3.5) found the sodium sulfate soundness results correlated better with field performance.

2) the number of cycles - As would be expected the soundness loss is proportional to the number of cycles used. Many agencies, including AREA, recommend the use of 5 cycles (3.7).

3) the void characteristics of the particles - Large voids and continuity of voids allow for greater buildup of the sulfate crystals and therefore may lead to increased loss in the soundness test.

4) mineralogy of the aggregate - The mineralogy of the aggregate may have a significant effect on aggregate particle strength characteristics and thus on "resistance" to soundness loss.

5) presence of fractured surfaces and sharp corners - An increase in the number of fractured surfaces and associated microscopic cracks and sharp corners increases the exposure surface of particles. The susceptibility of the particles to penetration and buildup of crystals increases correspondingly, and thus the soundness loss would more than likely increase.

c. Absorption

The voids and the continuity of the voids affect the results of the absorption tests. Slags and other coarse textured materials generally show higher absorption values than do materials such as basalt.

Peckover (3.1) feels the absorption test offers little in addition to the information obtained from freeze-thaw and soundness tests for the prediction of field degradation. He indicates a direct measure of porosity might be more useful.

d. Los Angeles Abrasion

The following are among the factors found to influence the results of the L.A. Abrasion test:

1) number of revolutions - The percent loss increases as the number of revolutions increases.

2) moisture condition during testing - The test may be conducted with either wet or dry aggregate. The standard procedure specifies dry aggregate but West, et al. (3.2) conducted tests on both conditions in the belief the wet test simulated more closely the field conditions. West, et al. (3.2) concluded the wet tests were more severe for basalt and dolomite but had little effect on the granite gneiss results.

3) uniformity of aggregate - The uniformity of the aggregate is supposedly measured by the ratio of the loss after 100 revolutions to the loss after 500 revolutions. A high ratio indicates the presence of soft material in the aggregate.

4) aggregate shape - Woolf (3.43) found flat and elongated particles cause an increase in loss, indicating particle shape also affects the test results.

5) mineralogy - The mineralogy of the particles also influences the results of the L.A. test. West, et al. (3.2) found that igneous rocks with high L. A. abrasion loss produced more fines than carbonate rocks with a similar abrasion loss. Sandstones in general also generated more fines than did the carbonates. Basalt was found to be unique in that as the Los Angeles number increased, the resistance to degradation increased. West, et al. (3.2) concluded the test was not suitable for predicting the field degradation of basalt.

e. Petrography

The results of petrographic analysis are greatly affected by the experience level of the petrographer and by the lack of quantitative analysis criteria. Because the nature of petrography is subjective, the results may have extreme variations.

f. Density

The density of ballast is affected by the specific gravity of the mineral, the apparent specific gravity of the particles, the gradation of the ballast, and relative compaction.

Powers (3.44) investigated the void ratio in different combinations of aggregate and found if size groups are similar, the size range governs the void content. Powers (3.44) also found that small void ratios are possible if large single size and small single size fractions are combined as well as if a well graded material is considered.

The specific gravities of ballast rock types depend on their geologic origins. Typical values for bulk specific gravities of some types of ballast particles are:

Basalt	2.85
Limestone	2.55
Dolomite	2.45
Gneiss	2.65
Blast Furnace Slag	2.40

It should be noted that variations of more than 20 percent for the specific gravity of a particular type of aggregate are not uncommon.

Peckover (3.1) recommends that ballast have a minimum specific gravity of 2.65. AREA recommends a minimum compacted unit weight of 70 to 100 pcf (1120 - 1600 kg/M^3) for slag ballast materials (2.1).

Heavy ballast is generally preferred to lightweight ballast because it is thought to offer more resistance to lateral movement (3.1).

g. Shape, Angularity, and Surface Texture

Differences in the shape, angularity, and surface texture of aggregate may be due to geologic factors and production methods.

Pit run gravels tend to be rounded, but because of their heterogeneous nature it is difficult to conclude much about the surface texture of gravels.

Crushed materials tend to be more angular than gravels, but the surface texture of crushed materials is primarily a function of the grain size of the material. Crushed granite has a rougher texture than does crushed limestone.

The shape of the particles may be influenced by the geologic deposit. Limestone obtained from fissured, well defined beds tends to crush into particles with one elongated axis. The method of crushing may also influence the final shape of the particles.

1. Flakiness

Peckover (3.1) cites the problem of "running rail" with uncrushed gravel ballast as an example of the effects of particle shape on performance. The CNR specifies ballast shall be "free from an excess of . . . thin or elongated pieces . . ." (3.1).

The definition of flaky (thin or elongated) particles varies but generally some ratio such as width to thickness, length to width, or least dimension to mean particle size is specified. Some examples of limiting ratios are 1.7 (British Standard), 3.0 (Swedish State Railways), and 4.0 (Ontario Ministry of Transportation and Communication) (3.1). The "flakiness problem" is further compounded because no standard exists for the allowable amount of flaky particles. The German Federal Railways believes a denser ballast is achieved if some "flaky" particles are included and therefore specifies both a minimum and maximum amount of such pieces (3.1).

Factors Affecting Strength Test Results

Many of the factors found to influence the quality of aggregate also influence the strength test results. In addition the type of test may influence the results.

a. California Bearing Ratio

Some of the factors influencing CBR results are:

1) <u>shape, angularity, and surface texture</u> - Townsend and Madill (3.45) used the CBR test to determine the effect of angularity on the strength of aggregate mixtures. They concluded the strength increased as the percent crushed fraction increased. In general, more angular and rough aggregate will exhibit higher CBR values.

2) density and gradation - The CBR values for a given type and gradation of aggregate are generally higher for higher relative levels of compaction. The more well graded materials generally have higher CBR values.

3) size of aggregate - Because the CBR mold is only 6 in. (15.2 cm) in diameter, all material larger than 3/4 in. (19 mm) is removed and replaced with an equivalent amount between 3/4 in. (19 mm) and the number 4 sieve (4.8 mm). Because some ballast is predominantly larger than 3/4 in. (19 mm) CBR test may be of limited applicability to ballast evaluation.

4) degree of saturation - If considerable fines are present, the CBR values after the samples are soaked may be appreciably lower than unsoaked CBR values.

5) type of aggregate - The effect of this factor is not well defined, but certain rock types (similar gradation and density) may affect the CBR value due to greater inherent particle strength.

b. Triaxial Test

Some of the more important factors that appear to affect the results of triaxial testing are:

1) maximum particle size - Marachi, et al. (3.19) studied the effects of maximum particle size on the shear strength of aggregate by using three different triaxial cells with 2.8, 12, and 36 in. (7.1, 76 and 91 cm) diameter specimens. The following conclusions were reached:

a. As maximum particle size increased, the angle of internal friction, ϕ, decreased. The effect was most pronounced at low confining pressures.

b. As maximum particle size increased the axial strain at failure increased (3.19)

Gray (3.20) concluded shear strength as determined by the Texas Triaxial Method increased as maximum particle size increased (similar gradations) and recommended the maximum size of aggregate should be the largest that can be handled without segregation.

2) shape, angularity, and surface texture - Gray, (3.20) studied crushed and uncrushed aggregate in the Texas Triaxial Method and found that in general, the strength of the crushed materials (rougher surface texture, more angular, etc.) was greater. Vallerga, et al. (3.46) tested uniform size gladd beads with different textures and observed the shear strength to increase as the roughness increased. Vallerga, et al. (3.46) also studied the effects of angularity. They concluded, for equal compactive efforts, the measured angle of internal friction increased only slightly as particle angularity increased although the effect of differences in density was not directly considered.

3) confining pressure - As confining pressure increases, in general, the maximum allowable deviator stress increases but the angle of internal friction decreases for a given aggregate. Ferguson and Hoover (3.47) investigated three well graded crushed limestones and found a tendency for lateral restraint to increase before the specimen reached minimum volume, and therefore the triaxial test method may not simulate properly the field confining pressure conditions.

4) density - The shear strength of a particular type and gradation of aggregate has been shown to increase as the density increases (3.20), Figure 3.5.

5) gradation - One of the most widely used equations for expressing gradation has been the Talbot Equation:

$$p = 100 \left(\frac{d}{D}\right)^n \qquad (3.6)$$

where: d = the sieve size in question,

p = the percent by weight finer than the sieve,

D = the maximum particle size, and

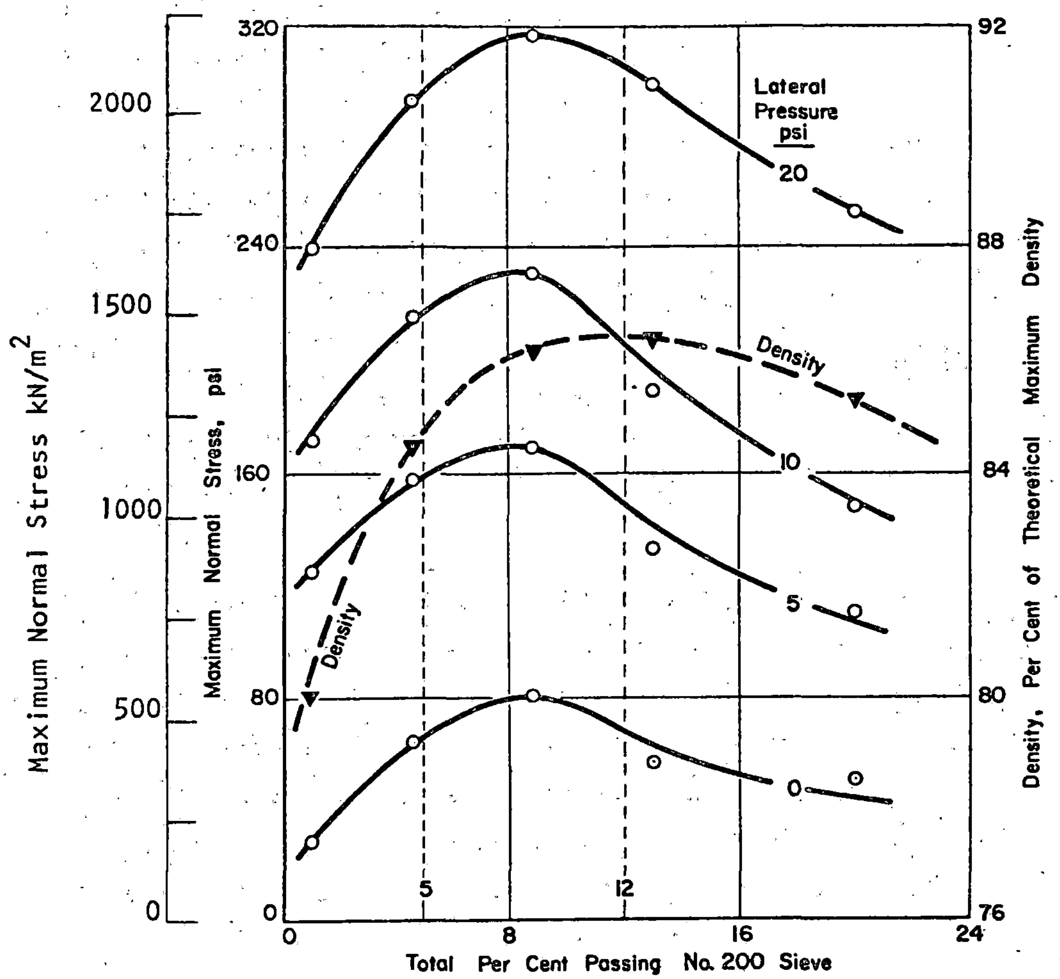

Figure 3.5. Relationship of Strength and Density for Material with Different Amounts Passing No. 200 Sieve (From Ref. 3,20)

n = the gradation exponent.

For highway base course materials, n is usually between 1/3 and 1/2. Maximum densities are achieved when n is equal to 1/2 (3.14, 3.24). Huang varied the gradation exponent and achieved maximum shear strength with n equal to 1/2, Figure 3.6. Gray (3.20) concluded the grading of aggregate "should be uniform from the maximum size down to and including the dust of fracture". Chung (3.48) found the effect of gradation on strength was negligible at low confining pressures and density.

6) amount and plasticity of fines - Texas Triaxial Test results reported by Gray (3.20) indicate maximum strengths are achieved if the amount of material finer than the number 200 sieve (0.074 mm) is between 5 and 12 percent (not more than 10 percent in areas where frost action may be a problem). This suggests that there is an "optimum fines content" for maximum shear strength. He also found that the strength is inversely related to the plasticity of the fines (3.20).

7.) specimen size - Two geometric considerations affect the triaxial test results: specimen length to diameter ratio and specimen diameter to maximum particle size ratio.

Bishop and Green (3.49) indicate the sample height should be a minimum of 2 to 2-1/2 times the diameter to minimize adverse end effects. The Texas Triaxial test specimen has a diameter of 6 in. (15 cm) but is only 8 in. (20 cm) high.

Studies by Holtz and Gibbs (3.50) and by Leslie (3.51) indicate that for triaxial testing the ratio of specimen diameter to maximum particle size should be in the range of 4 to 20. High proportions of the large size fractions require the ratio to be closer to the larger number.

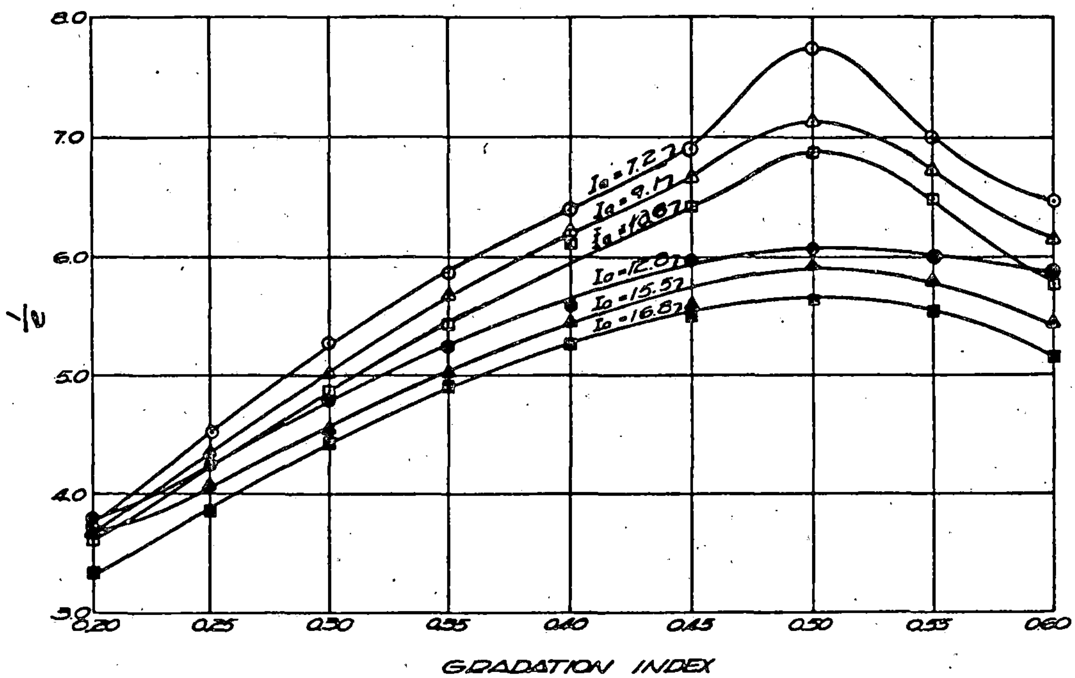

Figure 3.6. Relation between compaction characteristics and mixture gradation for soil-aggregate mixtures containing coarse aggregates of varying particle index (From Ref. 3.13).

8) rate of load (or strain) application - Variations in the triaxial test rate of axial strain from 1.8 to 0.086 percent per minute were shown to have no significant effect on the test results (3.50). Extremely fast rates of loading will probably increase the apparent strength.

b. Shear Box

Many factors affect the results of this test including: rate of deformation, geometric characteristics of particles, gradation, amount and plasticity of fines, density, degree of saturation, etc. The relative effect of these factors should be similar to that discussed for each under triaxial testing.

Among the other factors influencing the results of shear box testing are the following:

1) magnitude of displacement - Klugar (3.21) tested ballast in a shear box and found the displacement required to mobilize the angle of shearing resistance (tan ϕ) increased rapidly; practically no displacement was required to obtain a tan ϕ equal to 0.3, but 0.3 to 0.5 in. (7 to 12 mm) of movement was required to increase tan ϕ to values of about 0.8.

2) normal stress - Increases in the normal stress affect the results of the shear box test, but Pike (3.22) believes the effects are not pronounced.

Factors Affecting Repeated Load Test Results

Considerable research has been done to determine the effects of field conditions on the behavior of various granular materials. However, no standard test method has been developed, although the repeated load triaxial test has been used extensively.

a. Resilient Behavior

Several factors including the following influence the results of dynamic repeated loading tests of granular materials.

1) stress level - Both the degree of confinement and the repeated deviator stress level directly affect the values of the resilient modulus.

Granular materials show increases in the resilient modulus as the repeated deviator stress and/or continuing stress is increased (3.26, 3.27, 3.28, 3.29, 3.30, 3.31, 3.32, 3.33). Because of the above two influences, studies have been conducted to develop relationships between the resilient modulus and the total stress, θ ($\sigma_1 + 2\sigma_3$), or first stress invariant (3.26, 3.27, 3.28, 3.29, 3.30, 3.31, 3.32, 3.33, 3.52).

2) density - The effects of density have been studied but have not been well defined (3.27, 3.31). In general, the studies indicate the resilient modulus increases as the density increases, but the effect is not always pronounced nor consistent.

3) degree of saturation - For well-graded granular materials, it has been demonstrated that as the degree of saturation increases the resilient deformation increases and the corresponding resilient modulus decreases (3.36, 3.53).

4) specimen size - There has been little written about the effects of specimen height to diameter ratio and diameter to maximum particle size for the dynamic repeated triaxial test, but the effects should be similar to those for the static version of the test.

5) frequency and duration of load - Several investigators have demonstrated that throughout the range of application of loads to which ballast is subjected in the field, the duration and frequency of load has little influence (3.27, 3.31, 3.32, 3.33).

6) stress history - So long as the sample has not been overstressed, the resilient response measured after approximately 100 load cycles has been shown to be representative of the material properties throughout a complex stress history (3.26, 3.27, 3.31).

7) shape, angularity, and surface texture - The effects of shape, angularity, and surface texture have not been well defined. Hicks (3.27) varied the aggregate angularity and found the

resilient modulus increased as particle angularity increased. The effect of particle shape was demonstrated further by Allen (3.31), Figure 3.7.

b. Permanent Deformation

Many of the same factors which influence the resilient response of granular materials also influence the permanent deformation characteristics. Some of the more important factors are:

1) stress level - Barksdale (3.26) concluded the permanent deformation decreased significantly as the confining pressure increased. According to Barksdale (3.26) the rate of accumulation of permanent deformation at low repeated deviator stresses was proportional to the number of load applications, but at deviator stresses above a critical value, the rate of accumulation of plastic deformation increased with increasing numbers of load cycles.

Barksdale (3.26) modified the hyperbolic stress-strain law of Duncan and Chang (3.54) to predict the permanent strain as measured by triaxial tests on well graded materials and developed the following expression:

$$\varepsilon_p = \frac{(\sigma_1 - \sigma_3) / (K\sigma_3^{\,n})}{1 - \dfrac{(\sigma_1 - \sigma_3)\ (1 - \sin\phi)\ R_f}{2\,(C\cos\phi + \sigma_3 \sin\phi)}} \qquad (3.7)$$

where ε_p = permanent axial strain

$K\sigma_3^{\,n}$ = a relationship defining the initial tangent modulus, psi

C = cohesion, psi

ϕ = angle of internal friction, degrees

R_f = a ratio relating the stress difference at failure to the asymptotic value of stress difference.

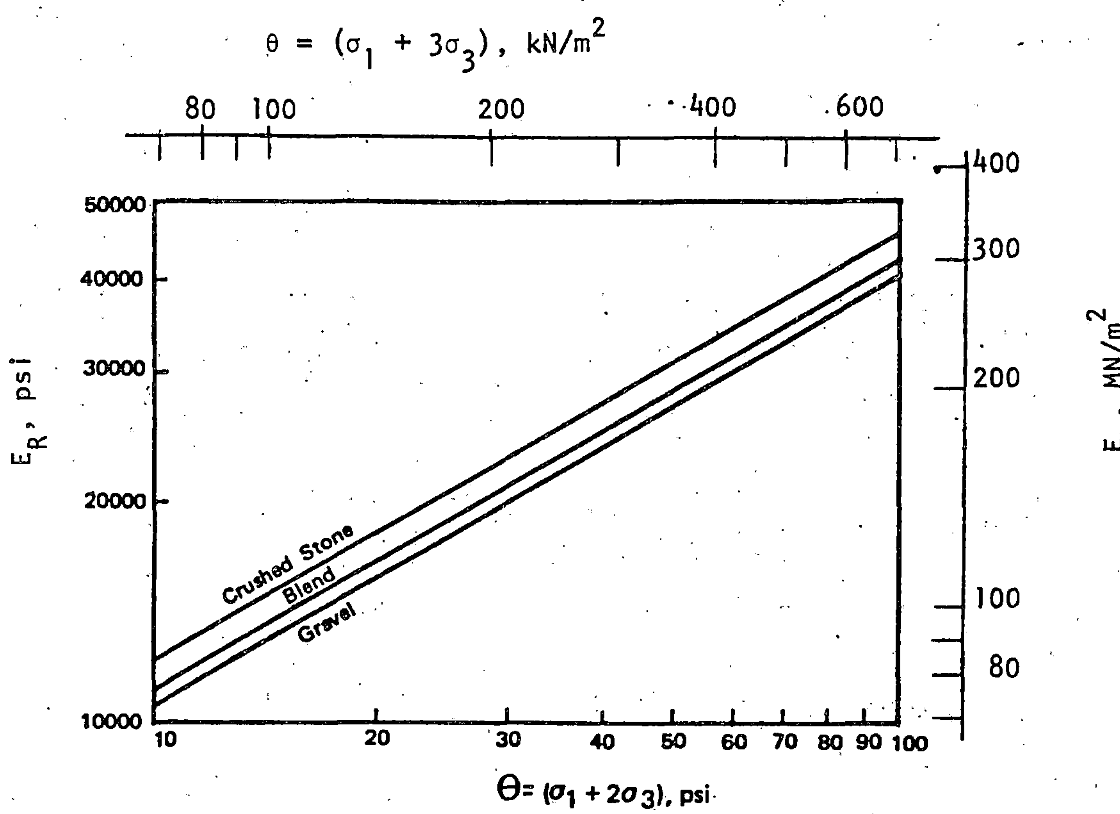

Figure 3.7. Effect of Material Type on the Relationship between E_R and θ, CCP Results, Low Density Specimens (From Ref. 3.31).

The importance of the deviator stress and the confining pressure is obvious. ORE (3.41) found the permanent deformation of ballast tested triaxially could be predicted by:

$$e_N = 0.082\ (100n - 38.2)\ (\sigma_1 - \sigma_3)^2(1 + 0.2\ \log N) \qquad (3.8)$$

where e_N = 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.

From the previous relationship the deviator stress and initial specimen porosity are shown to be extremely important factors during the first few load cycles.

2) degree of saturation - Haynes and Yoder (3.36) found that for well graded highway base course aggregates, the accumulation of plastic strain increased significantly as the degree of saturation increased.

3) density - Plastic strain accumulation is proportional to the initial porosity and is thus inversely proportional to initial density (3.41).

4) material type - Holubec (3.37) found the cyclic creep of a material was influenced more by the cohesion of the material than by the angle of internal friction. Increased cohesion resulted in significantly less permanent strain.

CHAPTER 4

FOUNDATION MATERIAL PROPERTIES

GENERAL

One of the important aspects of engineering design is to have an understanding and knowledge of the properties of the constituent materials in the system. Inherent in this knowledge and understanding is an appreciation for the factors influencing pertinent properties and the applicability or appropriateness of various property evaluation procedures.

Ideally, basic material response parameters should be known in order to dimension the system components. However, the required basic response parameters may not be well defined and even if they were, techniques for incorporating them into the design process may not be available. As a result increased dependence is placed on experience, judgment, and some measure of basic material properties in the design process.

Of primary interest in this section is a discussion of pertinent foundation material properties, methods of evaluating them, and factors affecting them.

PERTINENT FOUNDATION MATERIAL PROPERTIES

Foundation material properties that are of significance with respect to the design of a track support system include:

1. strength properties (compressive, shear, etc.)
2. elastic or resilient properties
3. plastic or permanent deformation properties

METHODS OF EVALUATING FOUNDATION MATERIAL PROPERTIES

Numerous methods and techniques have been developed for the purpose of evaluating pertinent properties of foundation materials. Some of the methods directly assess strength or deformation properties, and other test methods assess properties which correlate with certain strength properties.

Ideally, one would like to have a test method which could characterize strength properties or stress-strain properties for conditions closely simulating potential in-service conditions. In-service conditions of significance relative to property assessment include such factors as:

1. loading conditions (rate and duration of loading, number of loads, stress history, etc.)
2. temperature regime (temperature conditions as a function of time and space)
3. moisture regime (moisture conditions as a function of time and space)

<u>Static Strength Testing</u>

The importance of proper evaluation of the support capabilities of the soil material underlying a pavement structure has been recognized for many years as an important aspect of design. As recently as 15 to 20 years ago, tests used to evaluate soil support for pavement design purposes were static type tests or tests in which the load was applied at a relatively slow rate. It is acknowledged that the support value developed from static type tests does not represent the support conditions that exist in the soil under a pavement.

Common procedures for evaluating "static" soil support characteristics include:

1. plate load tests,
2. triaxial compression tests, and
3. California Bearing Ratio test

Most of these testing procedures have been standardized. Table 4.1 indicates the ASTM and AASHTO testing procedure designations for each.

a. Plate Load Tests

Plate load tests can be used to evaluate support. Plate diameter normally ranges from 12 in. (30 cm) to about 30 in. (76 cm) for these tests although the 30 in. (76 cm) diameter is used most often. In general, the procedure consists of loading the plate and measuring vertical deformation at the top of the plate. A non-repetitive or repetitive loading procedure can be used. The results can be used to calculate a "modulus of subgrade reaction", with the following equation:

$$k = p/\Delta \tag{4.1}$$

where:

k = modulus of subgrade reaction, pci

Table 4.1 Summary of Standard Procedure Designations for Pertinent Soil Strength Tests.

Testing Procedure	Testing Procedure Designation	
	ASTM	AASHTO
Plate Load Tests		
1. Repetitive	D-1195	T-221
2. Non-Repetitive	D-1196	T-222
Triaxial Compression	D-2850	T-234
California Bearing Ratio	D-1883	T-193
Unconfined Compressive Strength	D-2166	T-208

p = plate pressure, psi

Δ = vertical movement of plate, in.

Fairly extensive use is made of plate load test results in rigid pavement design. However, because of the time and expense associated with running this test, a minimal number of tests are often conducted.

b. Triaxial Compression Tests

The triaxial compression test in general, is used to determine the shear strength of a soil as a function of lateral confining pressure. In the test, even though only compressive stresses are applied, the specimen generally fails by shearing. The theory of shear strength is fairly complex and thus due to space limitations, it will not be discussed herein; however, most soil mechanics text books contain extensive discussion of this topic. Shear strength of the subgrade is not normally given extensive consideration in the design of a pavement system or railroad support system; thus, no further discussion will be devoted to it. It is believed that other measures of soil support will be more meaningful in analysis - design of track support systems.

c. CBR Test

The California Bearing Ratio test, commonly called the CBR test has been used extensively in pavement material and subgrade soil evaluation. It basically consists of a penetration test in which a standardized piston (3 in.^{2}) (19.4 cm^2) is forced into a prepared specimen (field testing can also be done). The load required to cause a 0.1 in. (2.5 mm) penetration is compared to the load required to cause the same penetration into a high quality crushed stone which is considered to have a CBR equal to 100. The ratio of these loads is used to calculate the CBR of the material.

The results from this test cannot be expressed directly as fundamental response parameters. Some correlations, however, have been made between dynamic E and CBR. In general, as shown in Figure 4.1, the dynamic E-value in psi ranges from about 500 to 3,000 time the CBR value with an average of about 1,500. (Dynamic E-value in MN/m^2 ranges about 5-20 times CBR with an average of about 10). Other attempts (4.1) have been made to correlate CBR with the resilient behavior of fine-grained soils but to data have proven unsuccessful.

Figure 4.2 illustrates typical CBR values for various soil materials.

d. Other Static Testing Procedures

Other testing procedures have been used for subgrade support evaluation, but on a much more limited scale. Some of these include the Hveem stabilometer, various vane shear tests, and certain penetrometer tests. However, the popularity and thus the experience with these have been limited. Thus, no discussion will be included herein relative to these procedures.

EVALUATION OF REPEATED LOAD CHARACTERISTICS OF SOILS

Many years of experience and observation of pavement performance have shown that unexplainable and premature failures periodically occur in some pavements. Many engineers and researchers have strongly suggested that the reason for those premature failures is related to the inadequate and "crude" methods of evaluating material properties.

With the advent of the computer, more complex theories have been developed to calculate stresses, strains, deflections, etc. occurring in pavement systems subjected to transient loading. As a result, the ability to handle more realistic evaluation of support conditions has evolved. In recent years, soil support characterization using repeated, dynamic testing has progressed substantially.

Currently, repeated load triaxial and complex modulus testing are the most common tests used to evaluate the repeated load response of fine-grained soils. Probably the more widely used of the two is the repeated load triaxial test.

a. Repeated Triaxial Testing

The first extensive work done in repeated triaxial load testing of fine-grained soils dates back 15 to 20 years, to Seed and his associates (4.2, 4.3, 4.4, 4.5, 4.6, 4.7, 4.8, 4.9, 4.10, 4.11, 4.12). The equipment and testing procedures developed have been modified slightly by other investigators, but the basic test remains about the same.

The foundation soil typically is subjected to a large number of rapidly applied loadings. The duration of an individual stress pulse may be on the order of 0.1 second. The duration of stress pulse can be closely controlled with typical repeated load triaxial testing equipment. Additionally, the repeated load triaxial testing procedure has the advantage that the confining

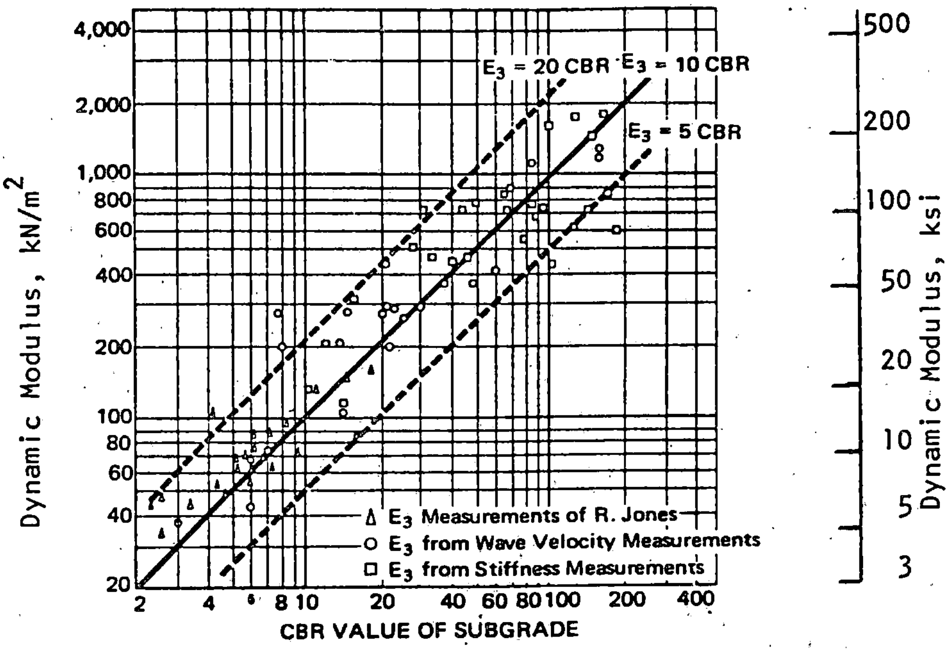

Figure 4.1. Relation Between Dynamic Modulus and CBR (From Ref. 4.27).

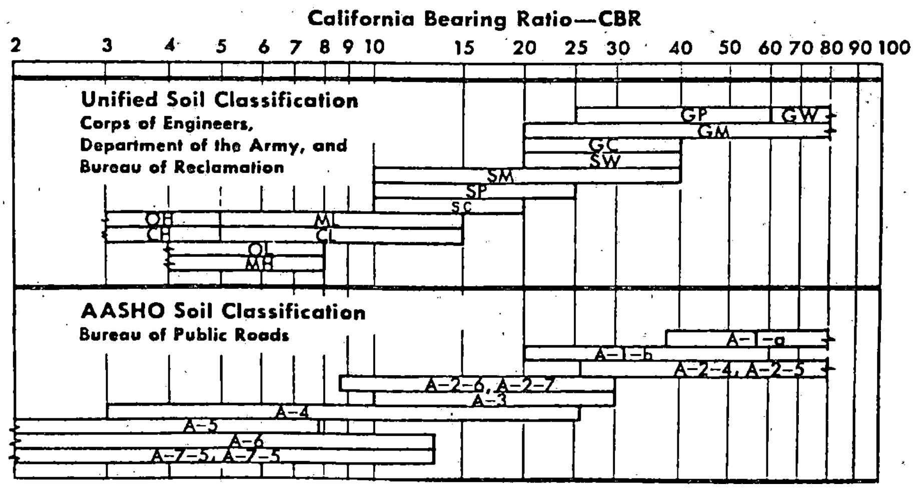

Figure 4.2. Typical Range in CBR for Various Soils (From Ref. 4.26).

pressure can be closely controlled, and a large number of stress applications may be applied to allow for evaluation of long term loading response.

The basic triaxial testing equipment used with fine-grained soils is pictured in Figure 4.3. Basically, the testing sequence consists of preparing a cylindrical specimen from the fine-grained soil, placing a rubber membrane around the specimen, placing the specimen in the test chamber, applying either a constant or repeated confining pressure, subjecting the specimen to a repeated axial load of controlled stress pulse, shape and duration, and monitoring the axial and diametral deformation. Typically, 20-30 loads per minute are applied. Pulse duration times typically range from 0.06 to 0.25 second, and pulse shapes vary depending on whether servo-hydraulic or pneumatic techniques are used for loading.

The types of data that can be obtained from this test include:

1. resilient modulus*
2. permanent strain behavior, and
3. dynamic Poisson's ratio

Figures 4.4, 4.5 and 4.6 illustrate typical resilient behavior and permanent strain data that can be obtained with the test.

b. Complex Modulus Testing

Papazian (4.13) has covered the fundamentals of the complex modulus concept. Basically, the complex modulus is determined by subjecting a sample to a sinusoidally applied steady state axial compressive stress while monitoring the strain which normally lags behind the stress by a phase angle, ϕ.

The general equation for complex modulus is:

$$|E| = \frac{\sigma_o}{\varepsilon_o} \tag{4.2}$$

where:

$|E|$ = complex modulus

σ_o = amplitude of sinusoidal axial stress

ε_o = amplitude of steady state or recoverable axial strain

Normally $|E|$ is determined at a given frequency of dynamically applied load (usually in the range of 1-20 cycles per second). Typical plots of complex modulus and phase angle are shown in Figure 4.7.

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*Resilient modulus = $\frac{\text{repeated deviator stress}}{\text{elastic or recoverable strain}}$

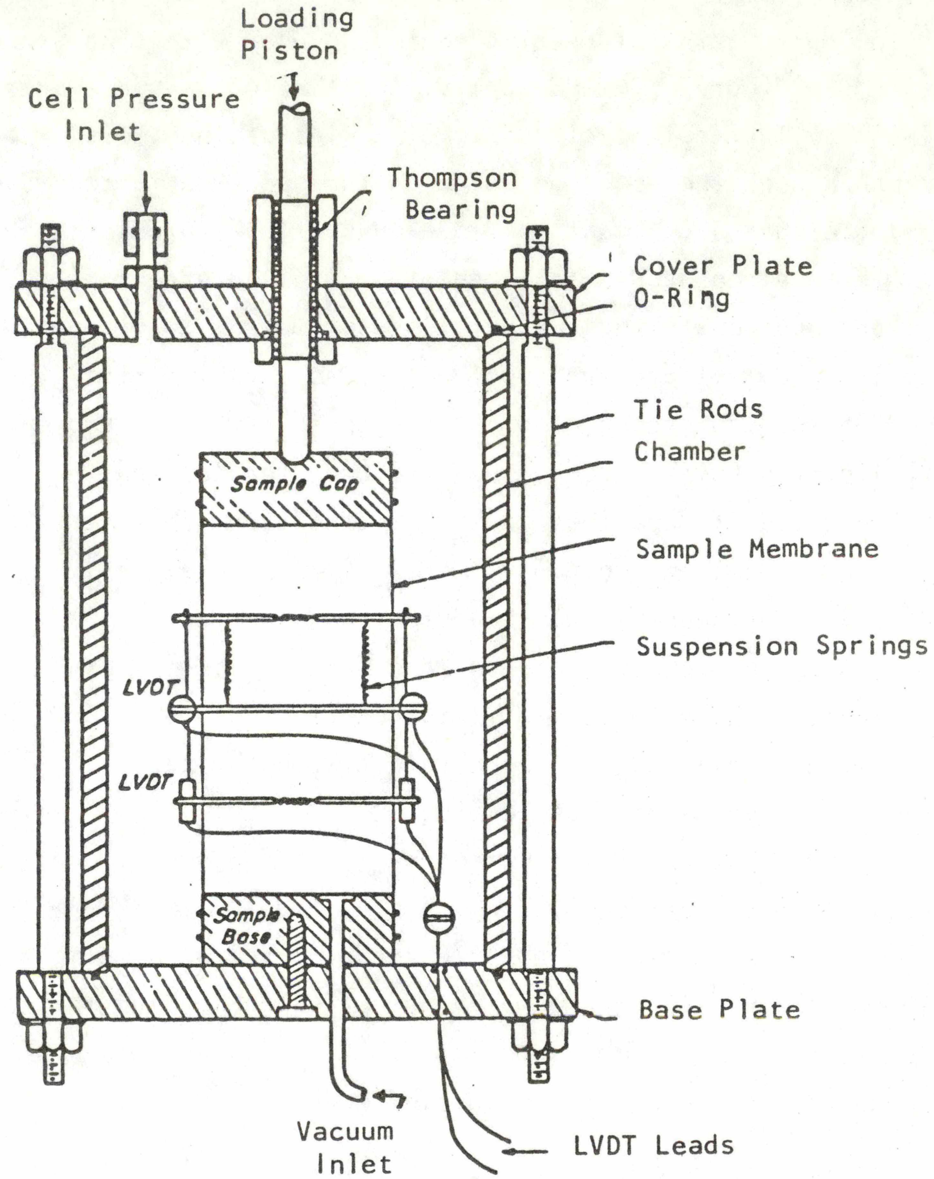


Figure 4.3. Typical Triaxial Testing Equipment

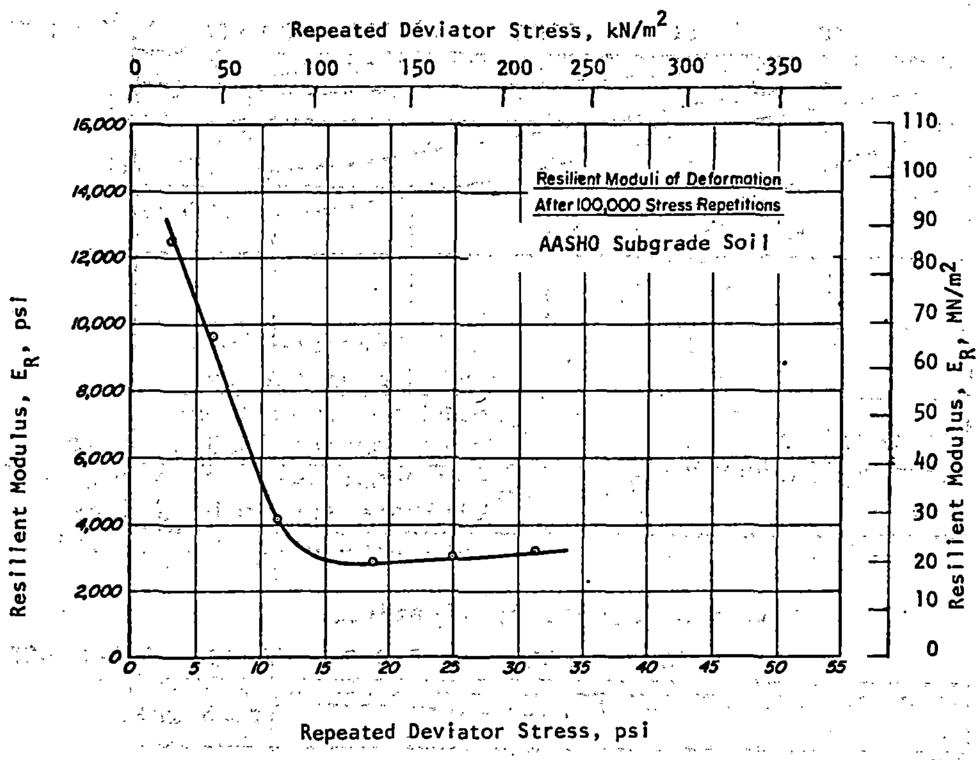


Figure 4.4. Effect of Repeated Deviator Stress on Resilient Modulus (From Ref. 4.6).

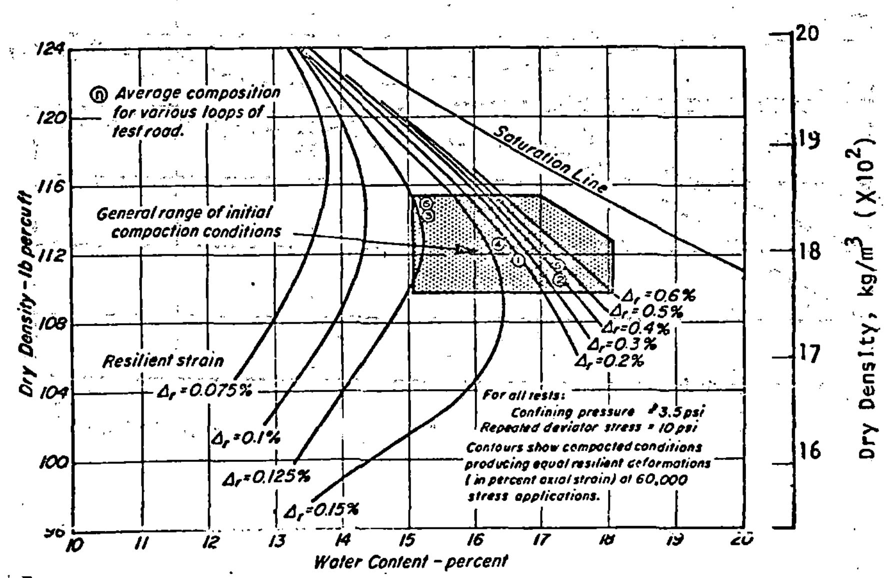
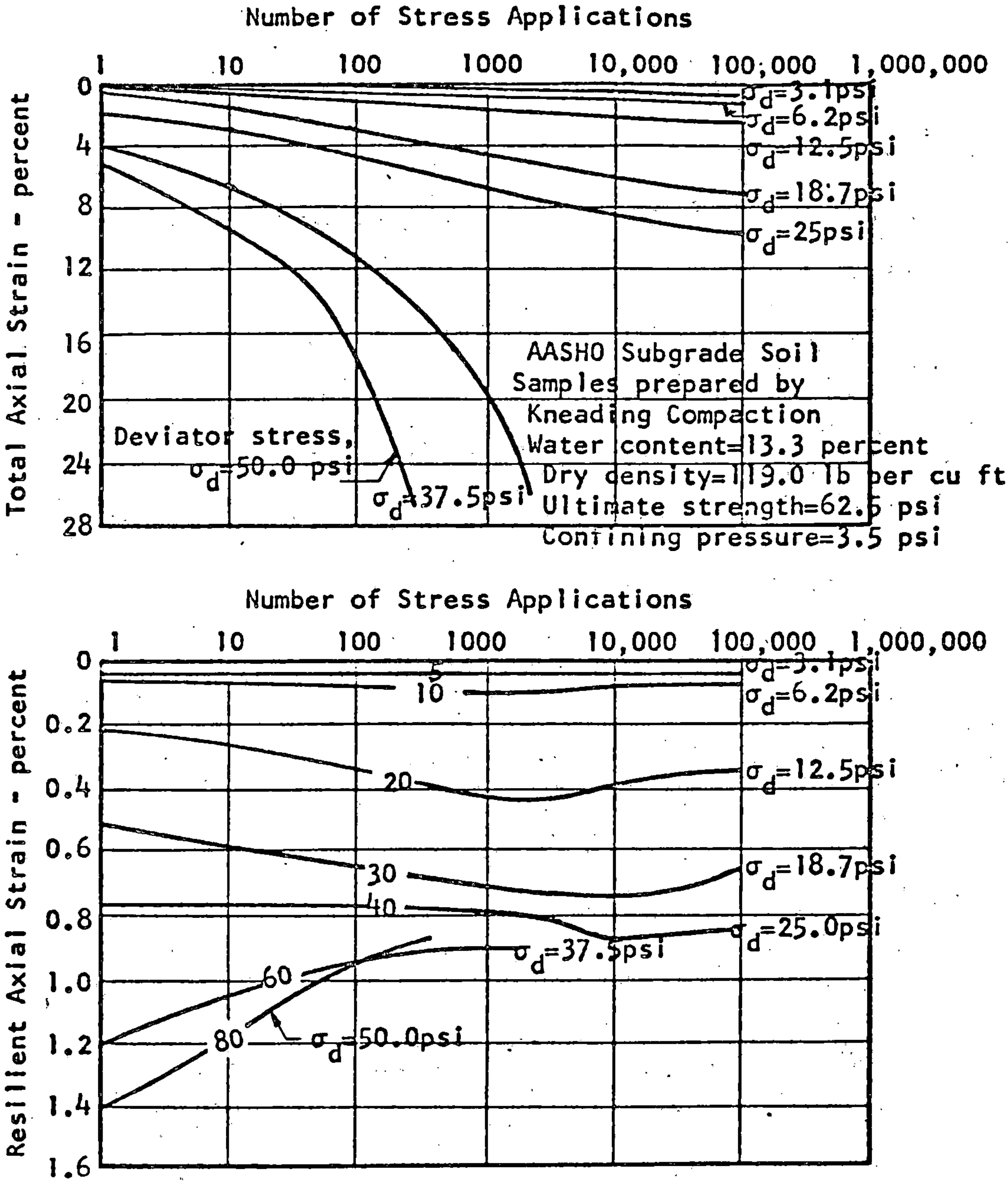


Figure 4.5. Effect of Moisture-Density on Resilient Behavior (From Ref. 4.6).



Note; 1 psi = 6.9 kN/m^2

Figure 4.6. Effect of Stress Level and N on Resilient and Total Axial Strain (From Ref. 4.6).

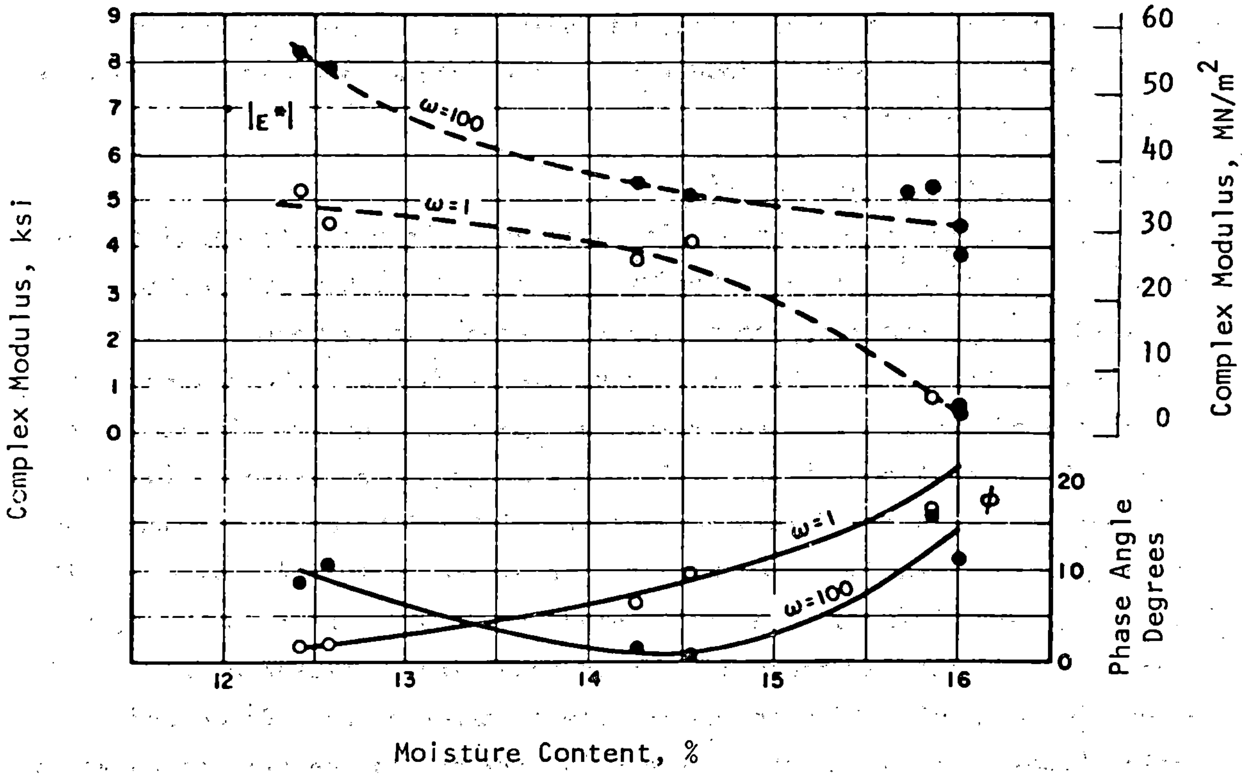


Figure 4.7. Effect of Water Content on Complex Modulus and Phase Angle for AASHO Road Test Subgrade Soil (From Ref. 4.14).

To date the complex modulus test has had limited use in the evaluation of fundamental material response parameters for fine-grained soils.

FACTORS AFFECTING FOUNDATION MATERIAL PROPERTIES

General

Numerous factors have been found to have a significant effect on pertinent foundation material properties, including such factors as loading conditions, moisture-density conditions, temperature, and certain inherent material characteristics.

Duration and Rate of Loading Effects

Most strength properties of fine-grained soils are influenced by the rate and duration of loading. Coffman, et al. (4.14) have shown that an increased loading frequency causes an increased complex modulus. Seed, et al. (4.15) have shown that the compressive strength of Vicksburg silty clay decreased about 30 percent when the time to failure was increased from 3 min to 600 min, Figure 4.8. Similarly, Casagrande and Shannon (4.16), found that for a range of loading time from 0.01-1000 sec., the unconfined compressive strength decreased by a factor of two. Olson and Kane (4.17) found that for a loading time that ranged from 0.003 sec to 100 sec., the unconfined compressive strength of Goose Lake Clay increased about 10 percent for each 10 fold increase in rate of deformation. Olson and Parola (4.18) found a general decrease in strength as the time or duration of load to failure increased.

Seed and Chan (4.2) found a definite effect of loading frequency and duration on the axial strain caused by repeated load triaxial testing of a silty sand, Figure 4.9. In general, it was found that increased duration of loading, and longer rest periods between loading, resulted in greater axial strains.

It is apparent that proper simulation of loading rate and duration is required for realistic evaluation of strength properties.

Number of Load Applications

For some types of strength tests, the effect of number of load applications is irrelevant, since only one load is applied. However, for

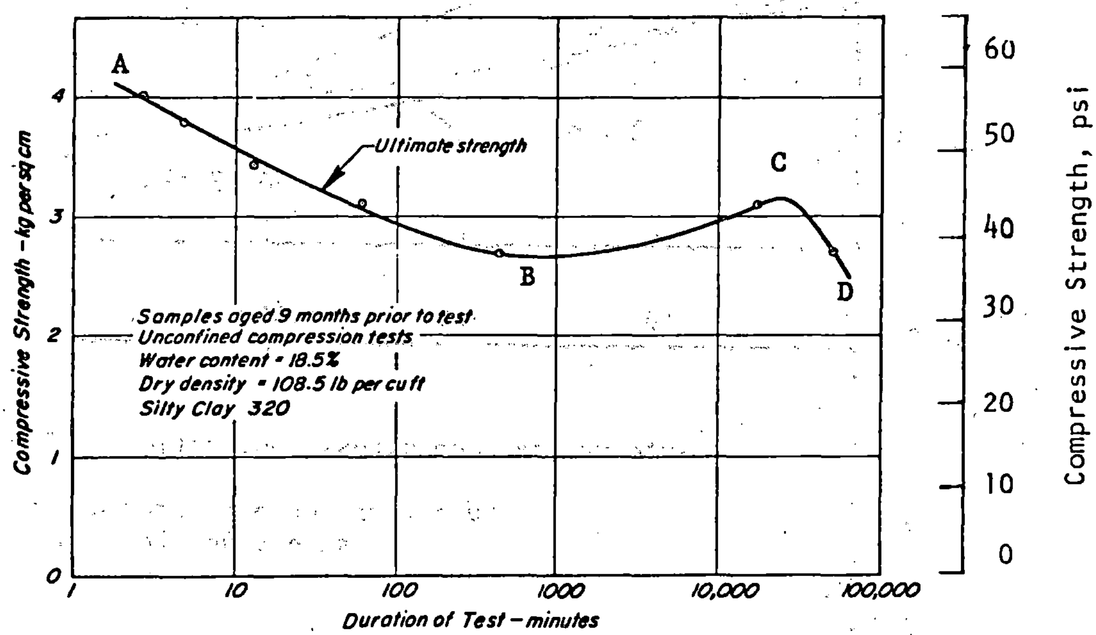


Figure 4.8. Effect of Loading Time to Failure on Compressive Strength (From Ref. 4.15).

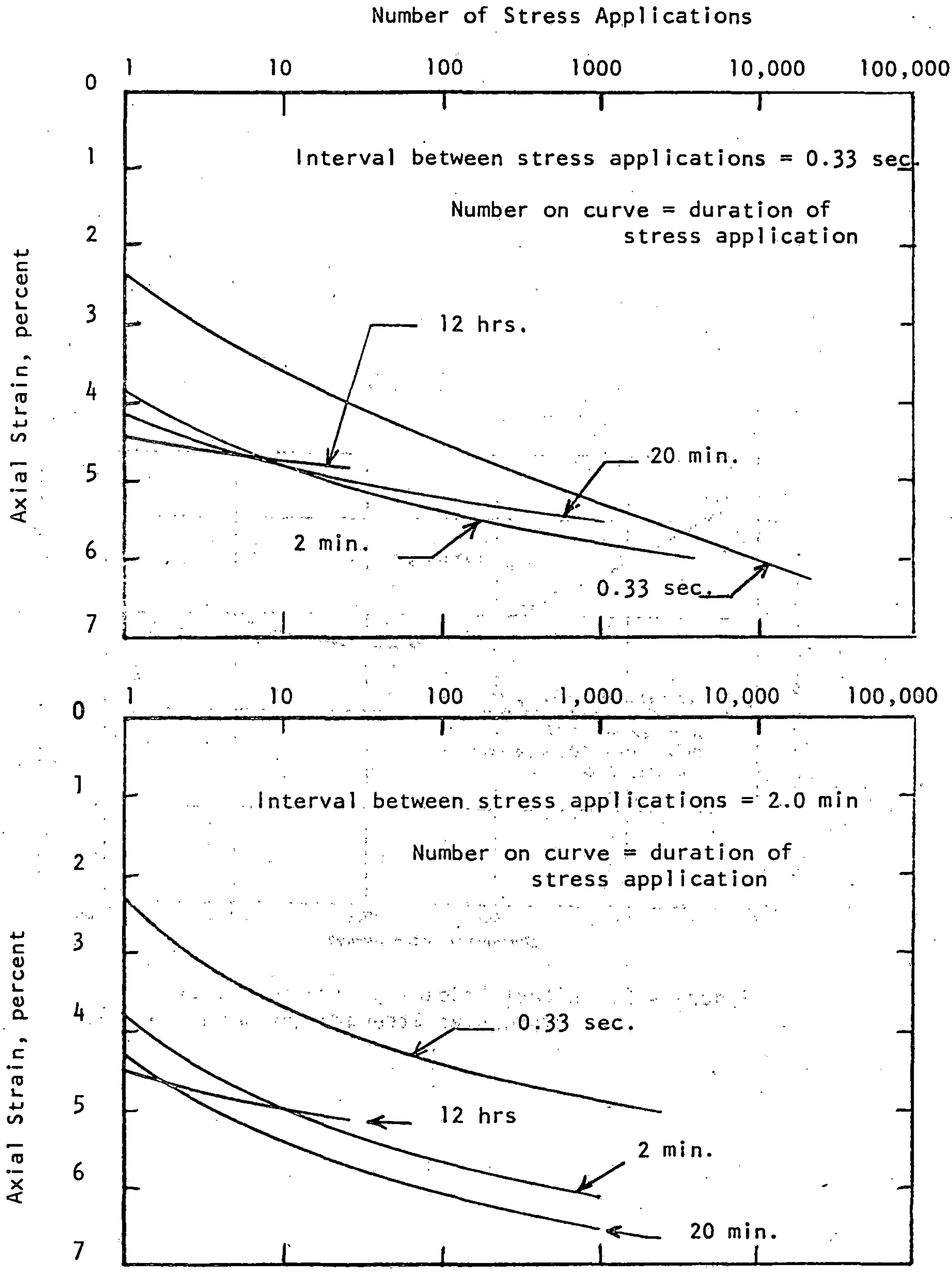


Figure 4.9. Effect of Duration of and Interval Between Stress Applications on Axial Strain (From Ref. 4.2).

procedures which use repeated load testing, such as the repeated load triaxial test, the number of loadings is a major factor influencing strength results.

The general trend is for an increasing number of load applications to correspond to increasing total strain, Figure 4.6. However, depending on factors such as degree of saturation, method of compaction, and inherent soil properties, the elastic or resilient component of the repeated axial strain resulting from the repeated axial load may either decrease or increase, Figure 4.10. Many investigators attribute this varied behavior to a thixotropic strength gain in "aged" or cured samples. It should be noted that after about 1000 load applications the elastic deformation noted for each load is about constant, Figure 4.11. Dehlen (4.19) found for purposes of testing about 200 applications at a given stress level were sufficient for "conditioning the soil."

In terms of permanent strain that accumulates during repeated loading, Monismith, et al. (4.20) have shown that permanent strain increases with N, but as N increases, the rate of permanent strain development decreases, Figure 4.12.

<u>Stress Level</u>

The effect of the magnitude of applied stress or stress level on certain strength properties of fine-grained soils is quite pronounced. As shown in Figure 4.13 as the stress level increases, the slope of the stress-strain curve normally decreases.

In terms of the effect of stress level on repeated load triaxial results, the data plotted in Figure 4.14 are characteristic of the "stress dependent" resilient behavior of fine-grained soils. As seen in the figure the resilient modulus decreases quite rapidly as the repeated deviator stress increases up to a σ_D value of about 6-8 psi (41-55 kN/m^2); then the rate of decrease of E_R rapidly diminishes with increasing σ_D.

Figures 4.15 and 4.6 depict the effect of stress level on total and resilient strains of a fine-grained soil subjected to repeated load triaxial testing. Typically, the total axial and permanent strains increase with an increase in stress level. Some studies (4.20, 4.32, 4.33) concerning permanent deformation characteristics of fine-grained soil have

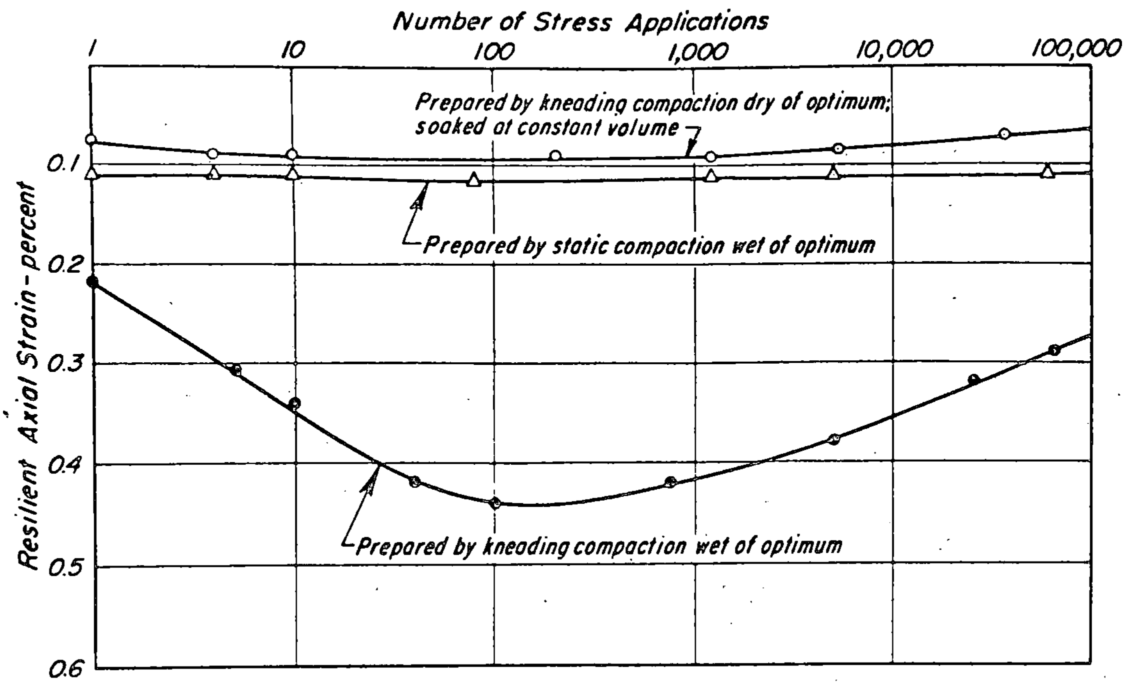


Figure 4.10. Effect of Compaction Method on Resilient Strain (From Ref. 4.11).

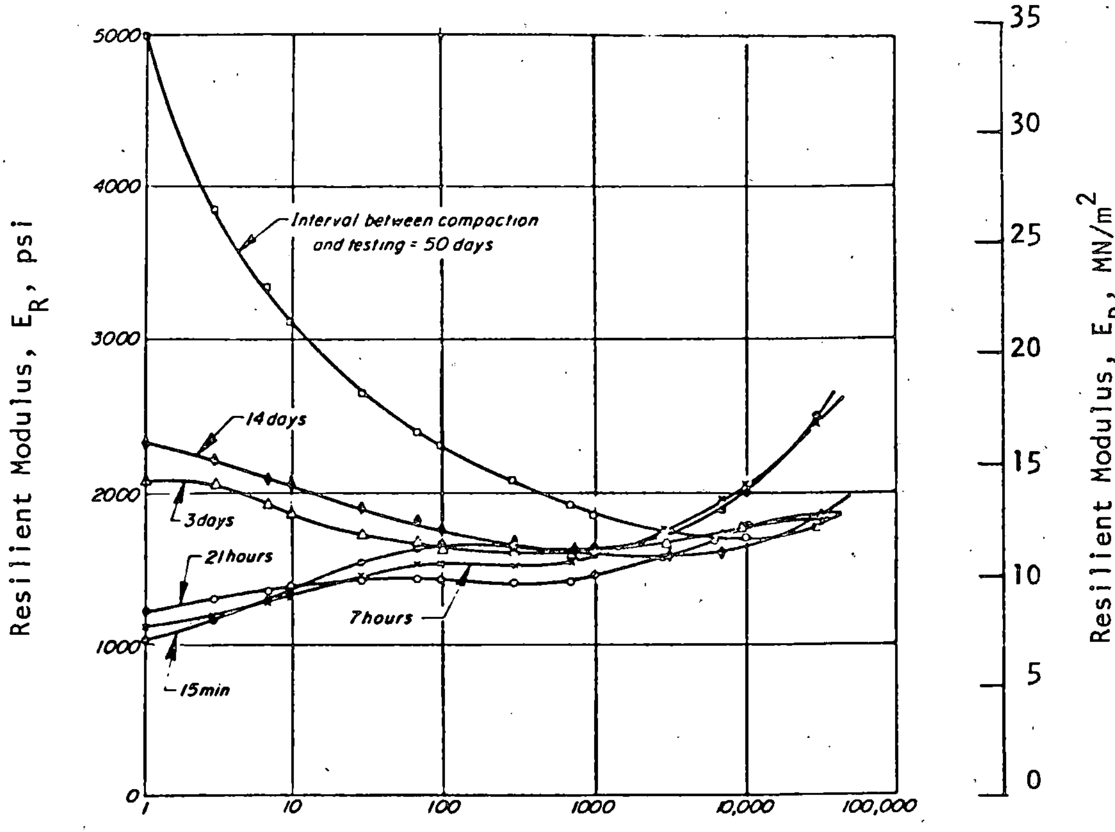


Figure 4.11. Effect of Aging on Resilient Modulus (From Ref. 4.6).

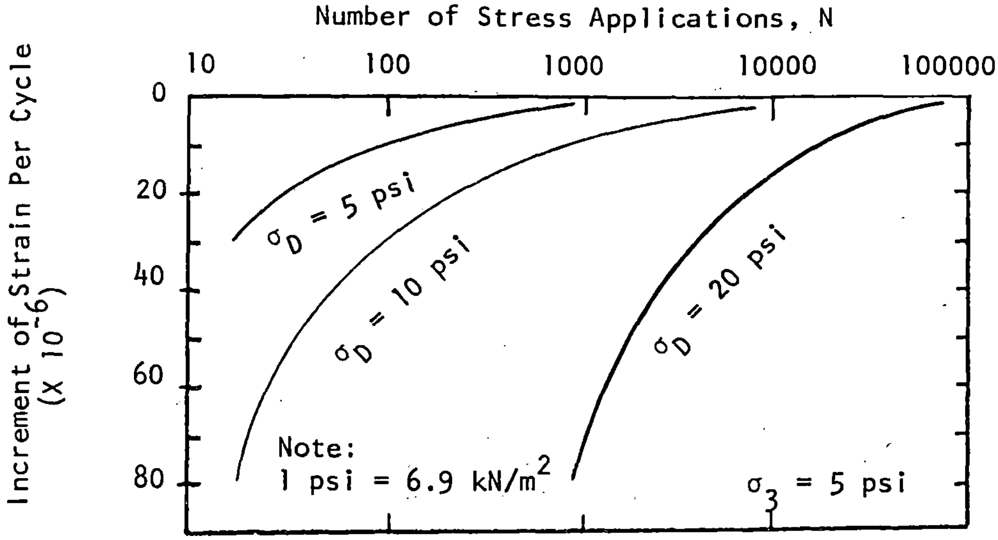


Figure 4.12. Change in Permanent Strain Accumulated Per Cycle as N Increases (From Ref. 4.20).

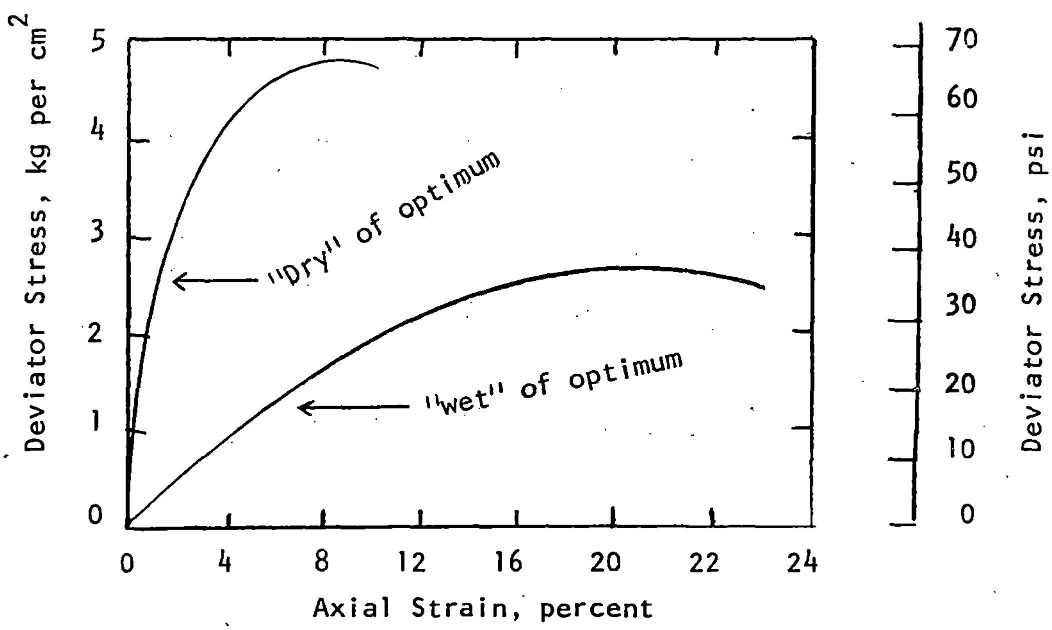


Figure 4.13. Typical Stress-Strain Curve for Fine-Grained Soil.

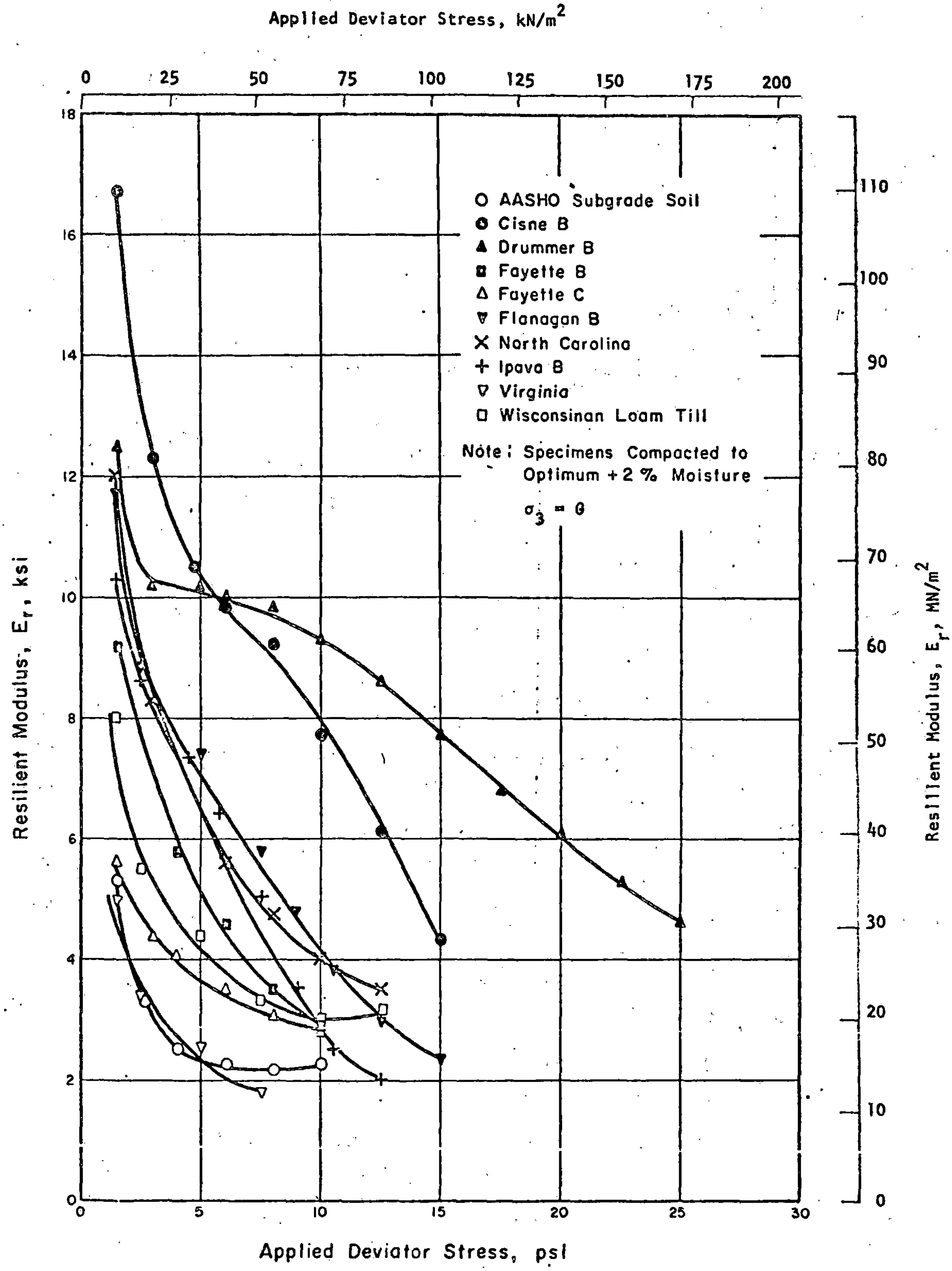


Figure 4.14. Typical Effect of Stress Level on Resilient Modulus (From Ref. 4.21.)

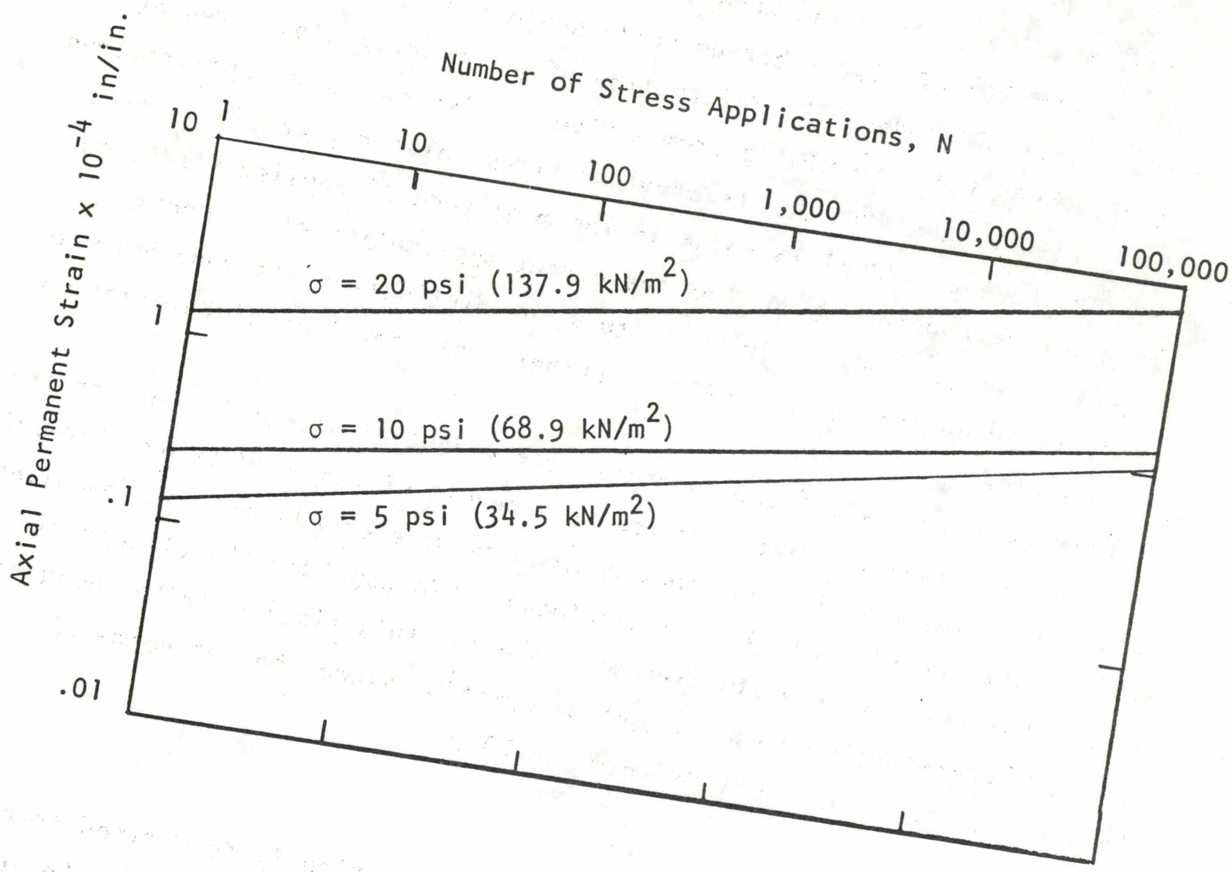


Figure 4.15. Effect of Magnitude of Applied Stress on Permanent Strain (From Ref. 4.20).

shown an apparent "threshold stress level." The "threshold stress" is defined as the stress level above which the permanent deformation of the soil under repeated loading is rapid and below which the rate of cumulative deformation from additional stress applications is very small.

Sequence of Loading or Stress History

The influence of stress history on repeated load strength properties is not well defined. In general, however, stress history appears to have a significant effect. Seed and Chan (4.3) studied the effect of stress history on the deformation of clay subgrade soils subjected to repeated triaxial loading. They concluded "The deformation of a compacted clay, even at constant composition, resulting from a given series of stress applications may vary widely depending on its previous stress history, and in general it appears that a gradual increase in the magnitude of applied stress may often cause less deformation than the direct application of a short sequence of wheel loads." In this study, no distinction was made between resilient and permanent deformation; rather, only total axial deformation was monitored.

Monismith, et al. (4.20) examined the effect of stress history on the permanent deformation that occurred as a result of repeated triaxial testing of a fine-grained soil. Numerous approaches were considered for "modeling" the results, but it was finally concluded: "To consider the effects of cumulative loading it would seem possible (at this time) only to bound the subgrade contribution to permanent deformation since the influence of stress history is not well defined" (4.20).

Confining Pressure Effects

As shown in Figure 4.16, typically the statically determined triaxial shear strength of an unsaturated fine-grained soil increases as the level of confinement increases. For repeated load triaxial testing, within the normal range of confining pressure encountered (σ_3 < 8-10 psi) (σ_3 < 55-69 kN/m^2), the resilient response of fine-grained soils does not appear to be significantly affected by magnitude of confining pressure, Figure 4.17.

The effect of confinement on the permanent deformation characteristics as evaluated in a repeated load triaxial test has not been extensively studied and thus is not well defined.

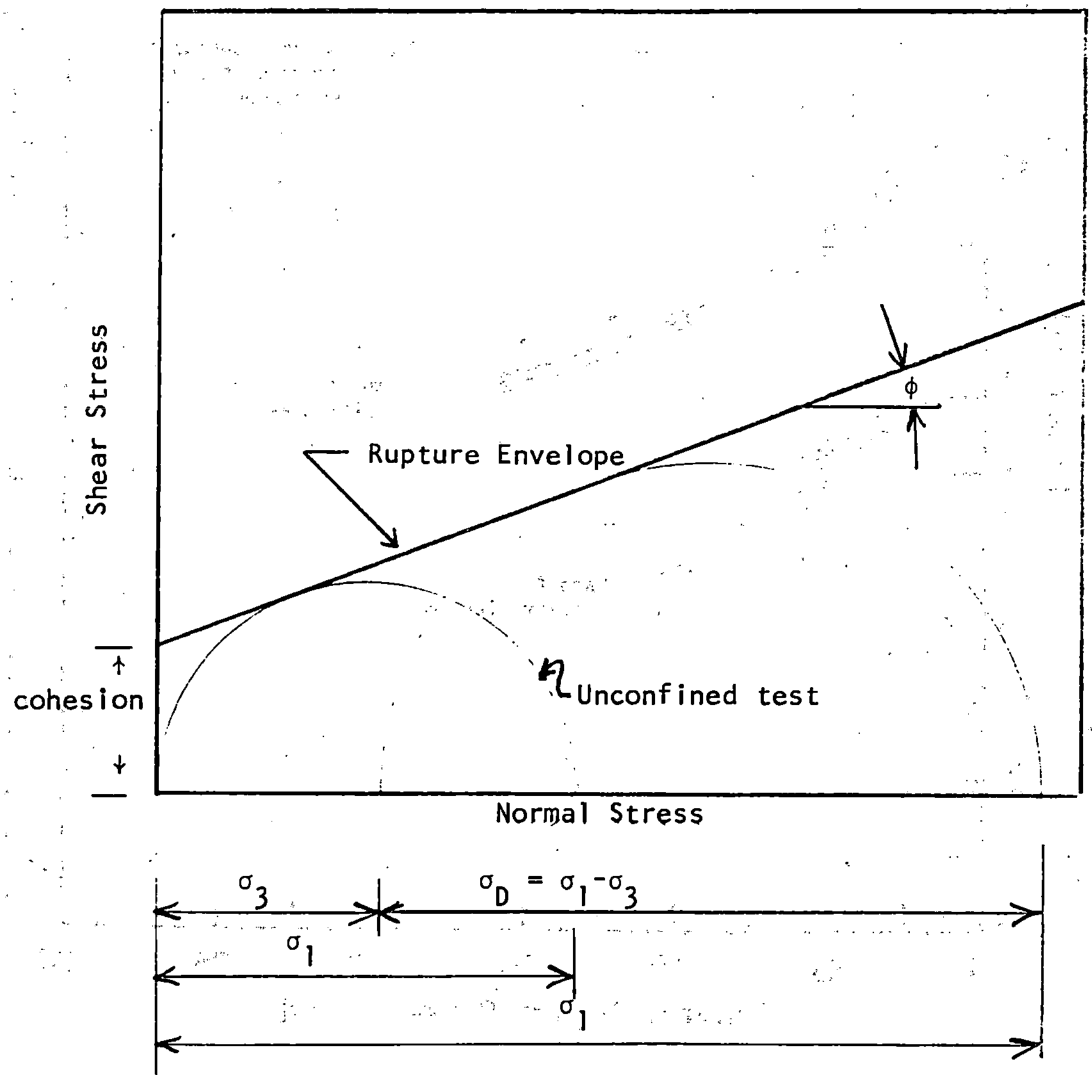


Figure 4.16. Typical Mohr Circle Representation of Triaxial Test Results

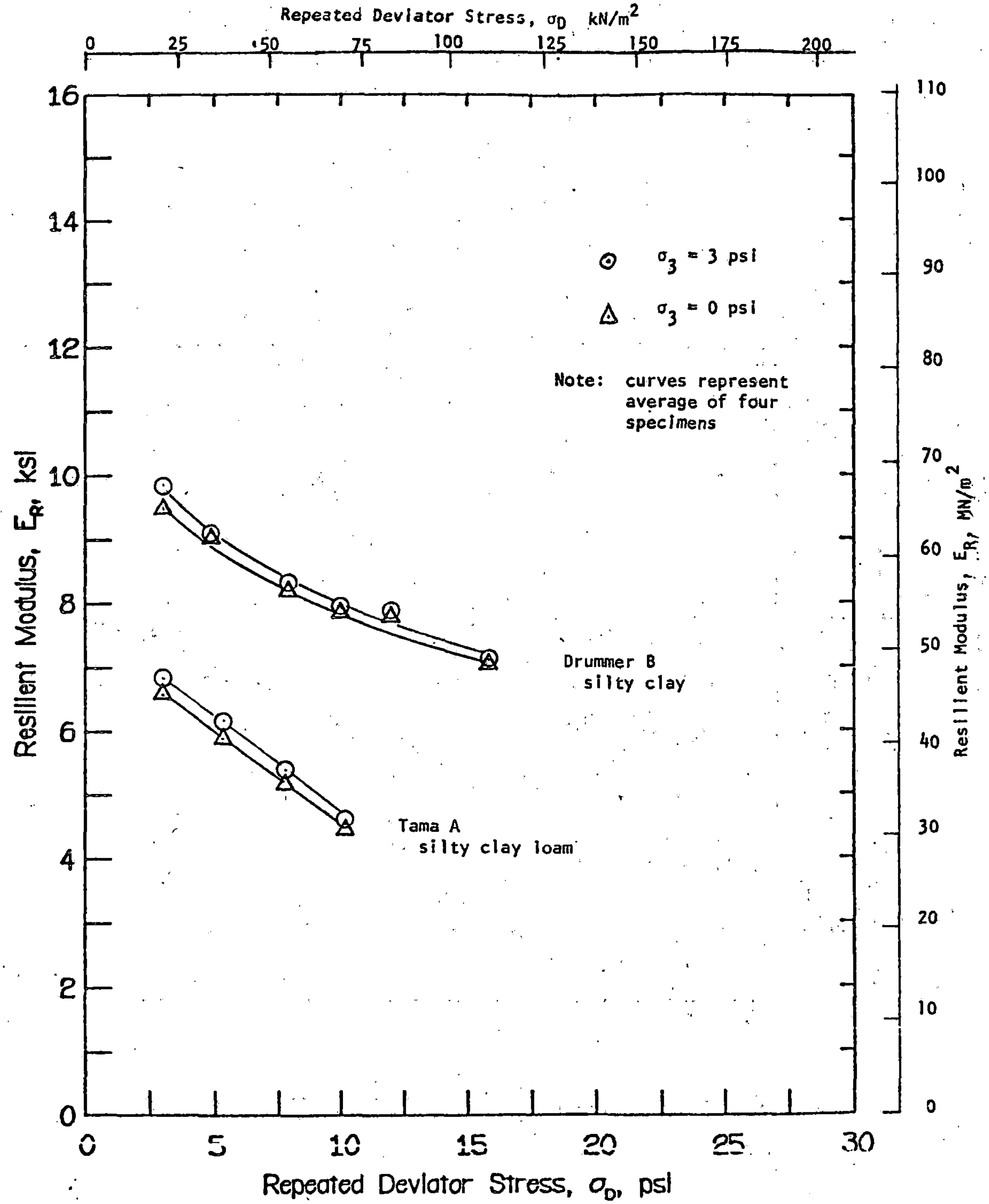


Figure 4.17. Effect of Confinement on Resilient Behavior (From Ref. 4.1).

Moisture and Density Effects

The effect of compaction moisture content and post-compaction moisture changes upon the strength and stiffness parameters of fine-grained soils is quite dramatic. Tremendous changes in strength/stiffness can result from only a few percent change in moisture content. Due to the influence of soil particle structural arrangement (flocculated vs dispersed) it is commonly accepted that initial compaction moisture-density conditions may have a substantial influence on subsequent changes in strength/stiffness characteristics.

As shown in Figure 4.18 initial compaction density-moisture conditions have a substantial effect on the loss in CBR due to soaking. Similarly, Figure 4.19 shows the effect of density-moisture on soaked compressive strength.

Extensive studies of the resilient behavior of fine-grained soils conducted by Robnett and Thompson (4.21) and others (4.6, 4.7, 4.11) have shown the detrimental effect of moisture on the repeated load resilient modulus. Figures 4.20 and 4.21 depict the influences of moisture content, density, and degree of saturation on the resilient behavior of various fine-grained soils. Results from the studies conducted by Robnett and Thompson (4.1) indicate that fine-grained soils display various degrees of "moisture sensitivity" depending on certain inherent characteristics of the soils.

Much of the strength change displayed by fine-grained soils can be explained by changes in "soil suction". As shown in Figure 4.22 as moisture increases soil suction is reduced. As shown in Figure 4.23 the resilient modulus of fine-grained soils is closely related to soil suction; a reduction in E_R is related to a reduction in soil suction.

It is significant to point out that the general tendency for clayey soils to exhibit higher E_R values than do silty soils (in the range around optimum moisture content) corresponds to the higher soil suction values of clayey soils.

Thus, it is reasonable to conclude that the repeated load resilient response of fine-grained soils will be affected detrimentally by moisture increases or by increases in degree of saturation. The absolute magnitude of change in resilient modulus caused by a 1 percent increase in soil

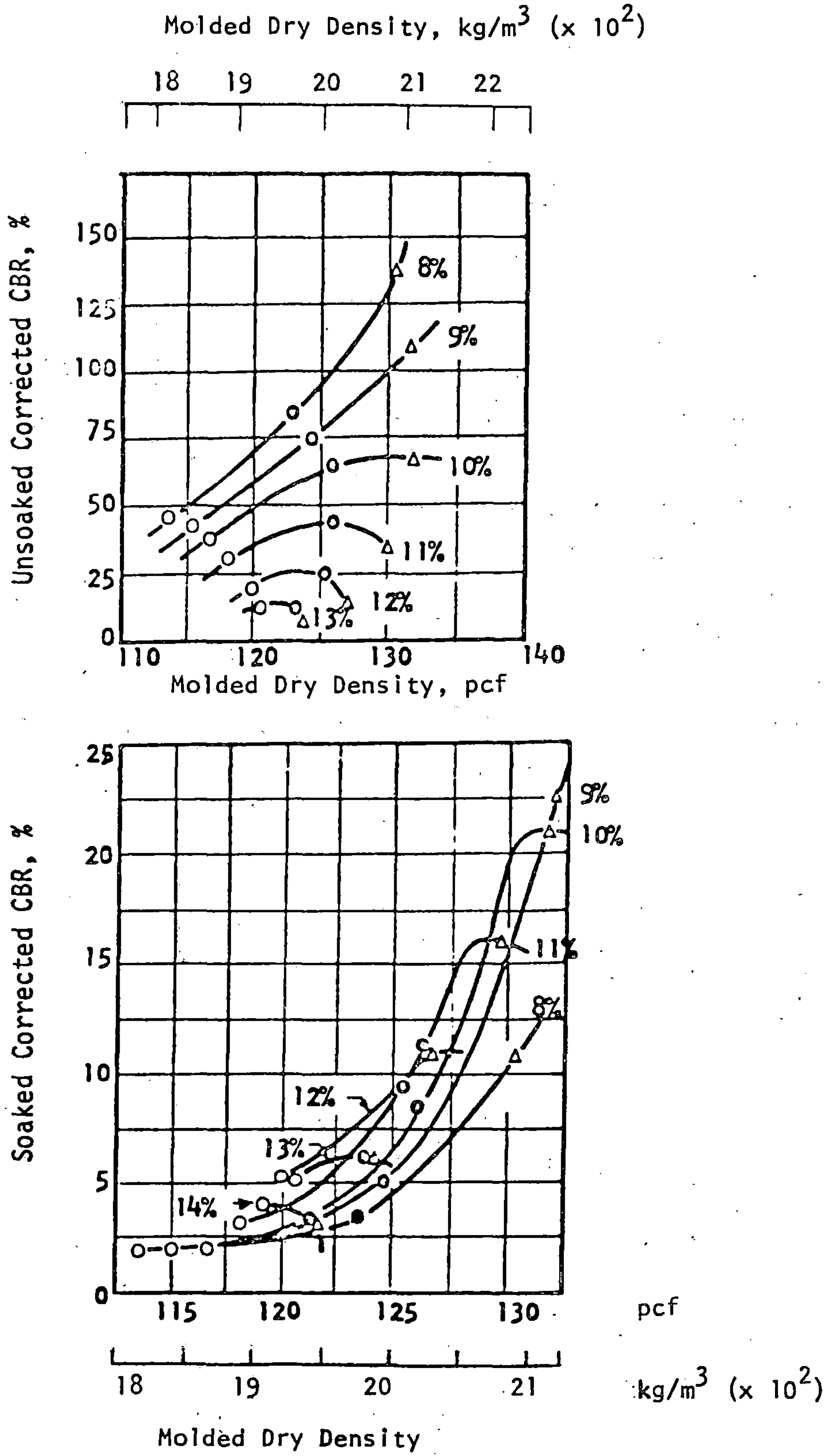


Figure 4.18. Typical Effect of Moisture-Density on Unsoaked and Soaked CBR.

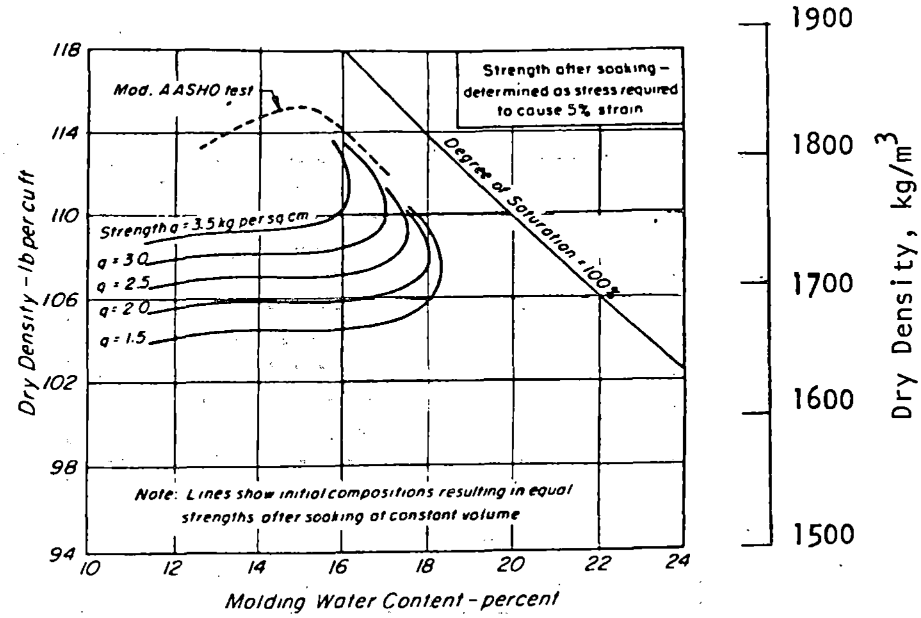


Figure 4.19. Effect of Moisture-Density on Compressive Strength (From Ref. 4.28)

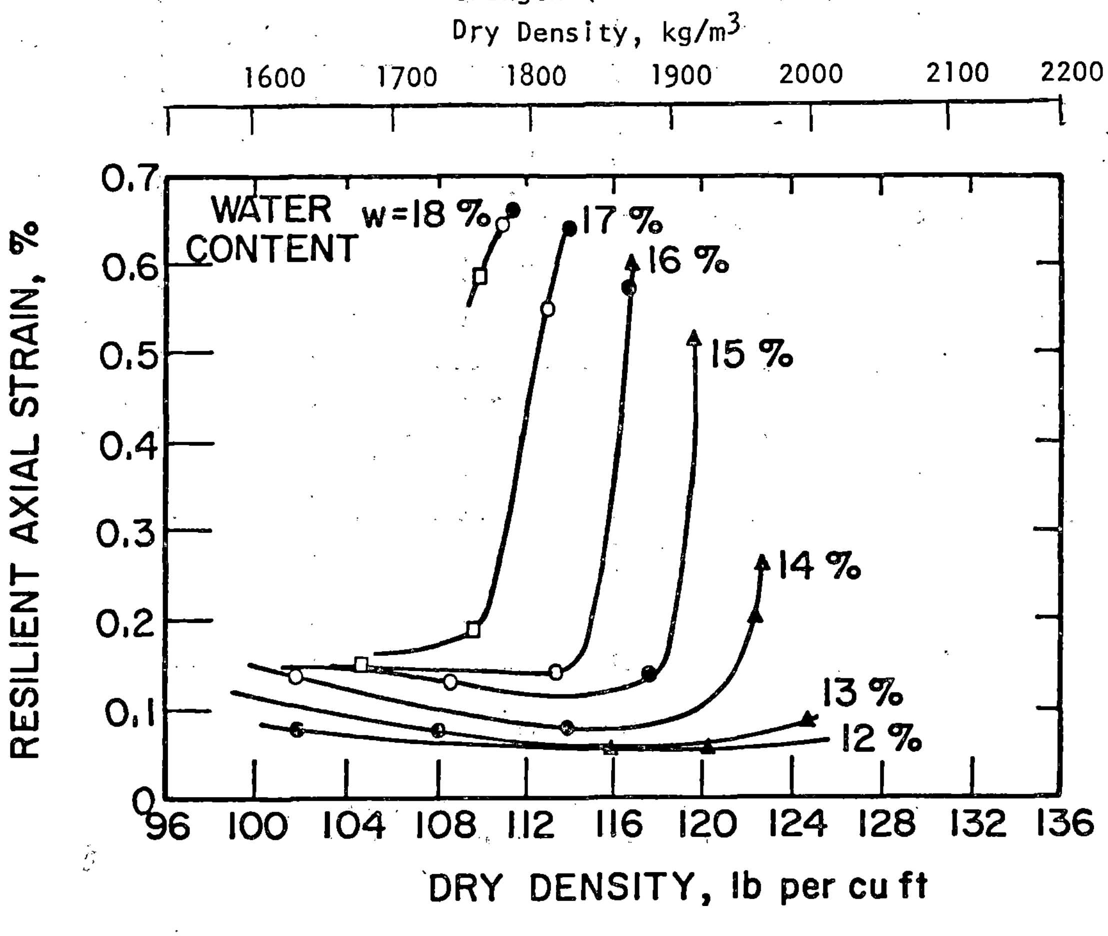


Figure 4.20. Effect of Moisture-Density on Resilient Behavior (From Ref. 4.6).

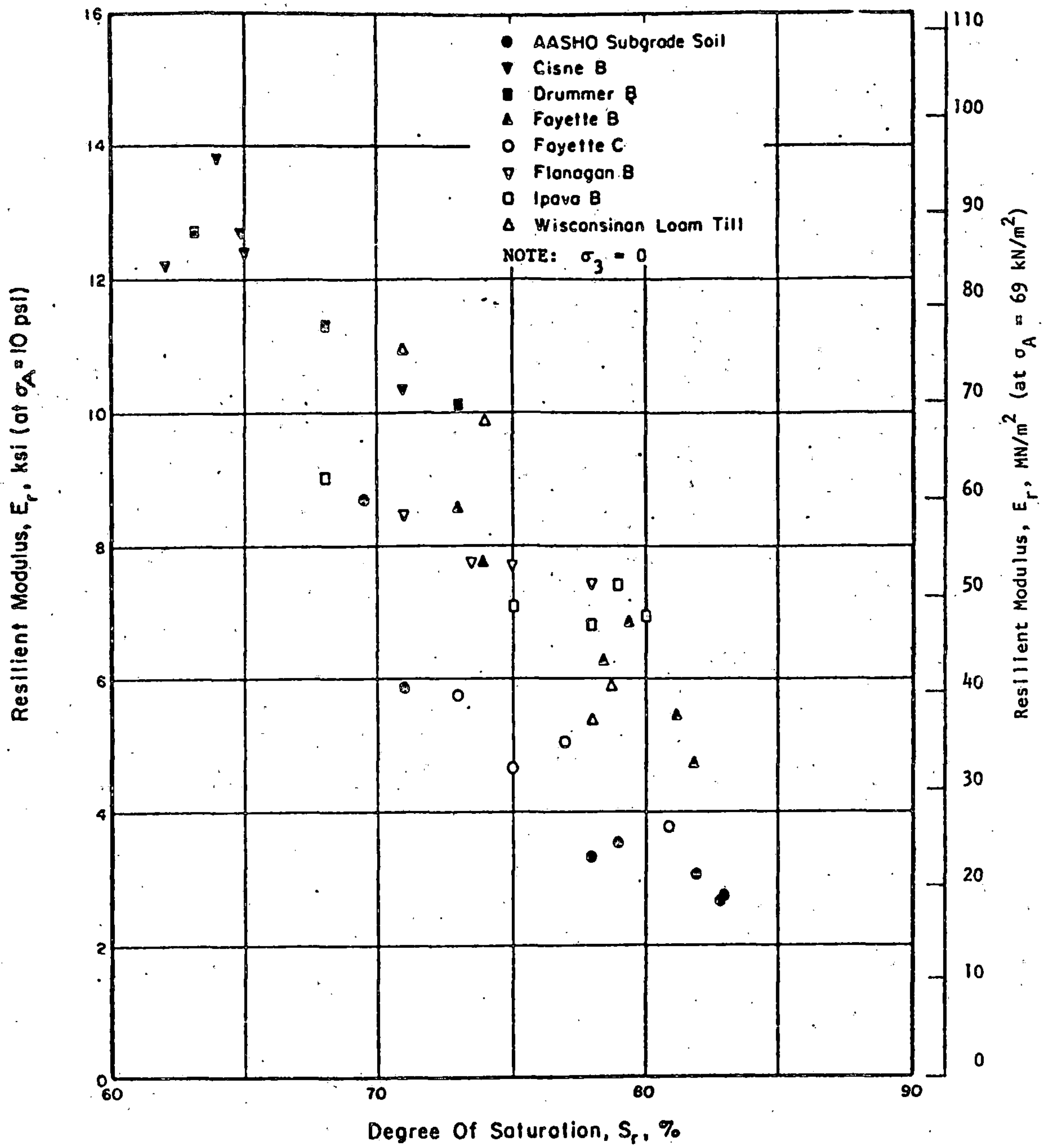


Figure 4.21. Effect of Degree of Saturation on Resilient Modulus (From Ref. 4.21).

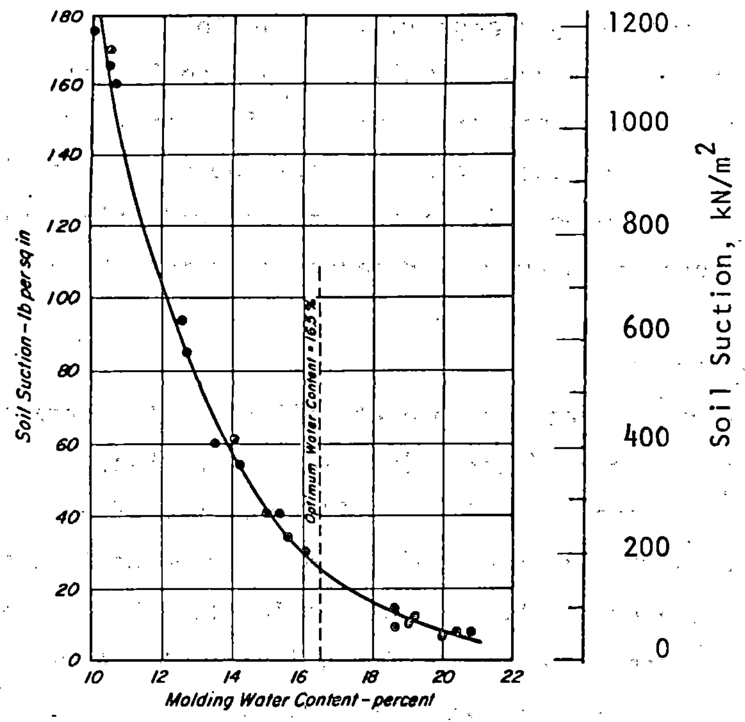


Figure 4.22. Effect of Moisture on Soil Suction (From Ref. 4.29).

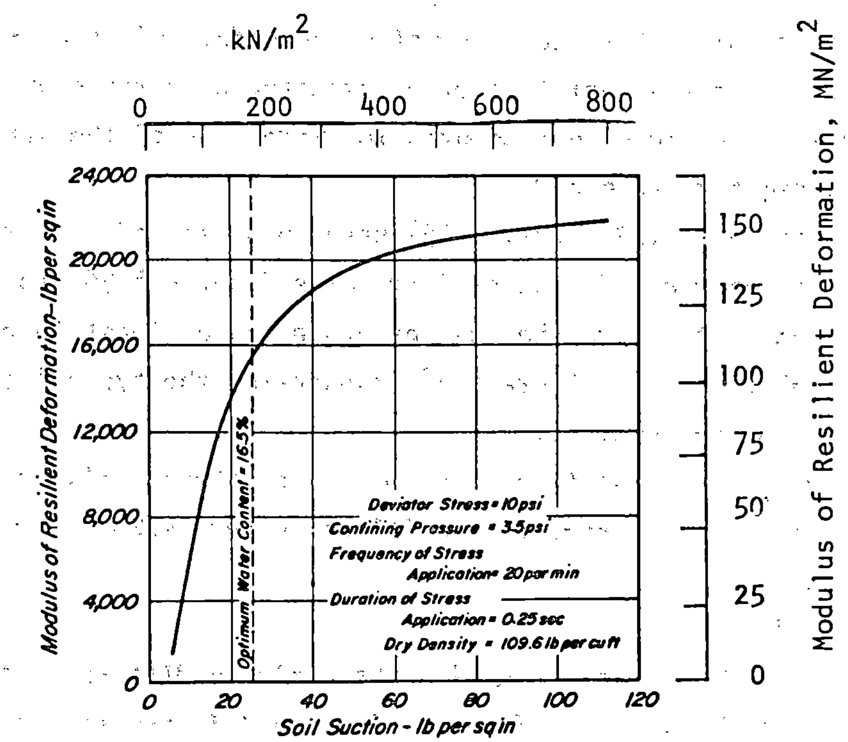


Figure 4.23. Effect of Soil Suction on Modulus of Resilient Deformation (From Ref. 4.29).

moisture content will not be constant for all soils but will vary depending on the accompanying change in soil suction.

The effect of changes in moisture and density conditions on the permanent deformation characteristics of fine-grained soils has not been well defined. It is reasonable to assume however, that increased moisture contents, higher degrees of saturation, and reductions in "soil suction" will lead to greater accumulated permanent deformation.

Temperature Effects

Temperature can have a moderate effect on the strength and stiffness parameters of unfrozen fine-grained soils. Perhaps this stems from the influence of temperature on the "soil suction". Figures 4.24, and 4.25 illustrate the typical effect of temperature on certain strength and stiffness parameters. Thus, it can be implied that if temperature affects soil suction, then temperature also will affect the modulus of resilient deformation as illustrated in Figure 4.23.

Freeze-thaw action can have a substantial effect on strength and stiffness parameters of fine-grained soils. Pagan and Khosla (4.22) have reported the results of a study in which the effect of freeze-thaw on the rheological characteristics of a compacted clay was examined. Complex moduli, creep, and unconfined compression were some of the strength/stiffness parameters examined.

Other studies (4.23, 4.24, 4.25) have shown the effect of F-T on the resilient behavior of fine-grained soils Figures 4.26, and 4.27.

The effects of freeze-thaw can be explained at least in part by the "soil suction" concept. Figure 4.28 illustrates the typical trend of soil suction and F-T cycling.

Some strength and stiffness parameters of frozen soils are substantially higher compared to the unfrozen conditions, Figure 4.25.

Effect of Soil Type

Fine-grained soils may exhibit substantially different strength characteristics due to inherent variations in soil properties such as placticity, clay and silt contents, organic matter content, clay mineralogy, etc. For example, as shown in Figure 4.2, there may be a substantial range in the CBR values of various fine-grained soil materials.

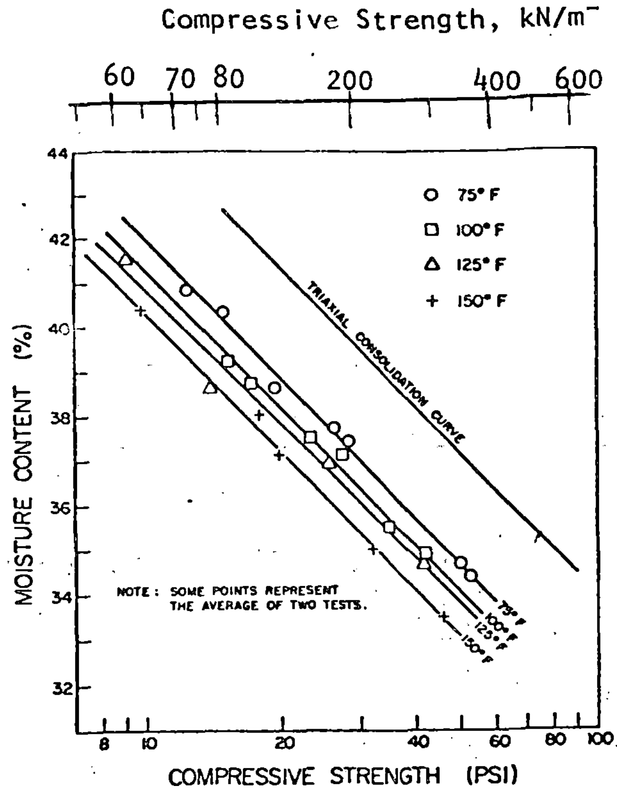


Figure 4.24. Effect of Temperature on Compressive Strength (From Ref. 4.30).

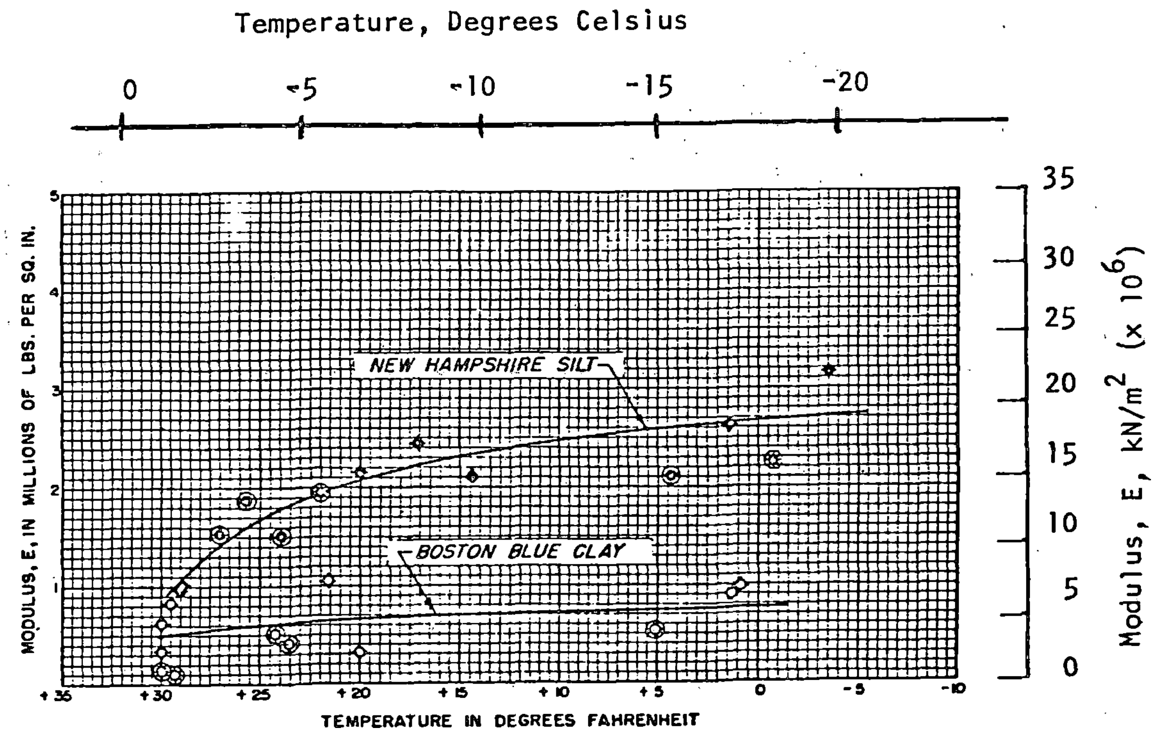


Figure 4.25. Effect of Temperature on Modulus of Deformation (From Ref. 4.31).

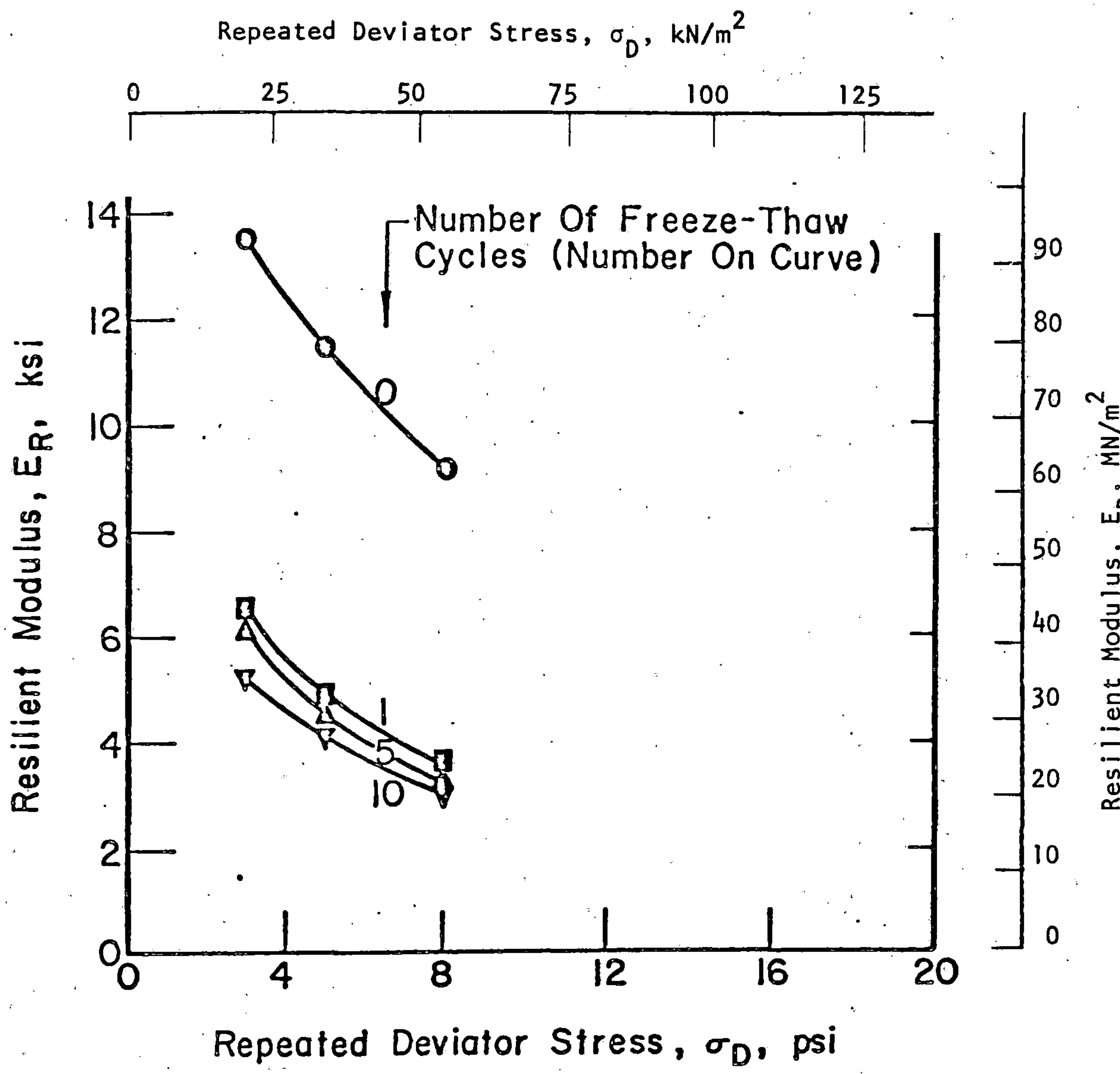


Figure 4.26. Effect of Freeze-Thaw Cycling (closed system) on the Resilient Behavior of a Clayey Soil (From Ref. 4.25).

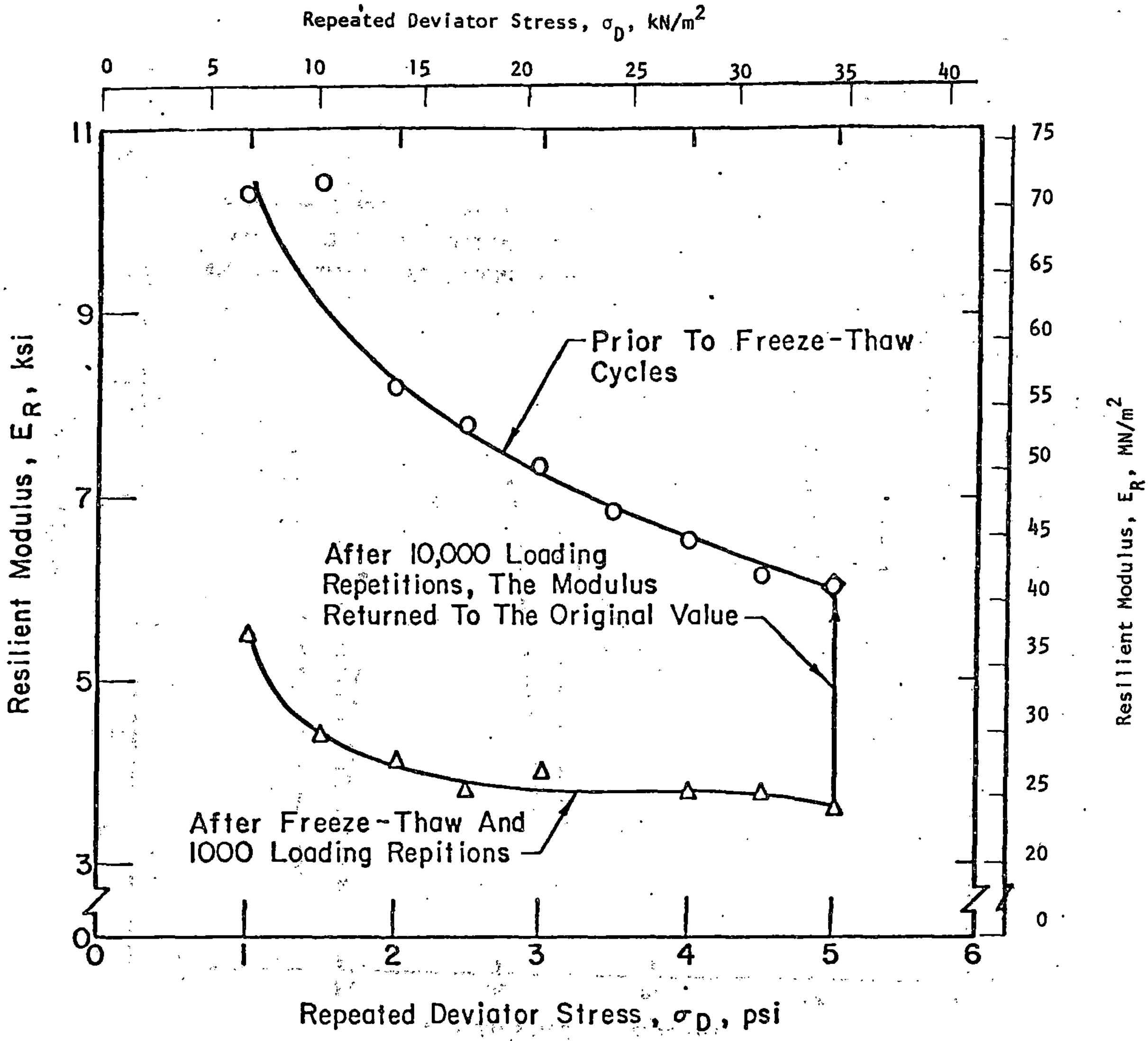


Figure 4.27. Resilient Modulus Test Results Before and After Freeze-Thaw for Undisturbed Regina Clay (From Ref. 4.23).

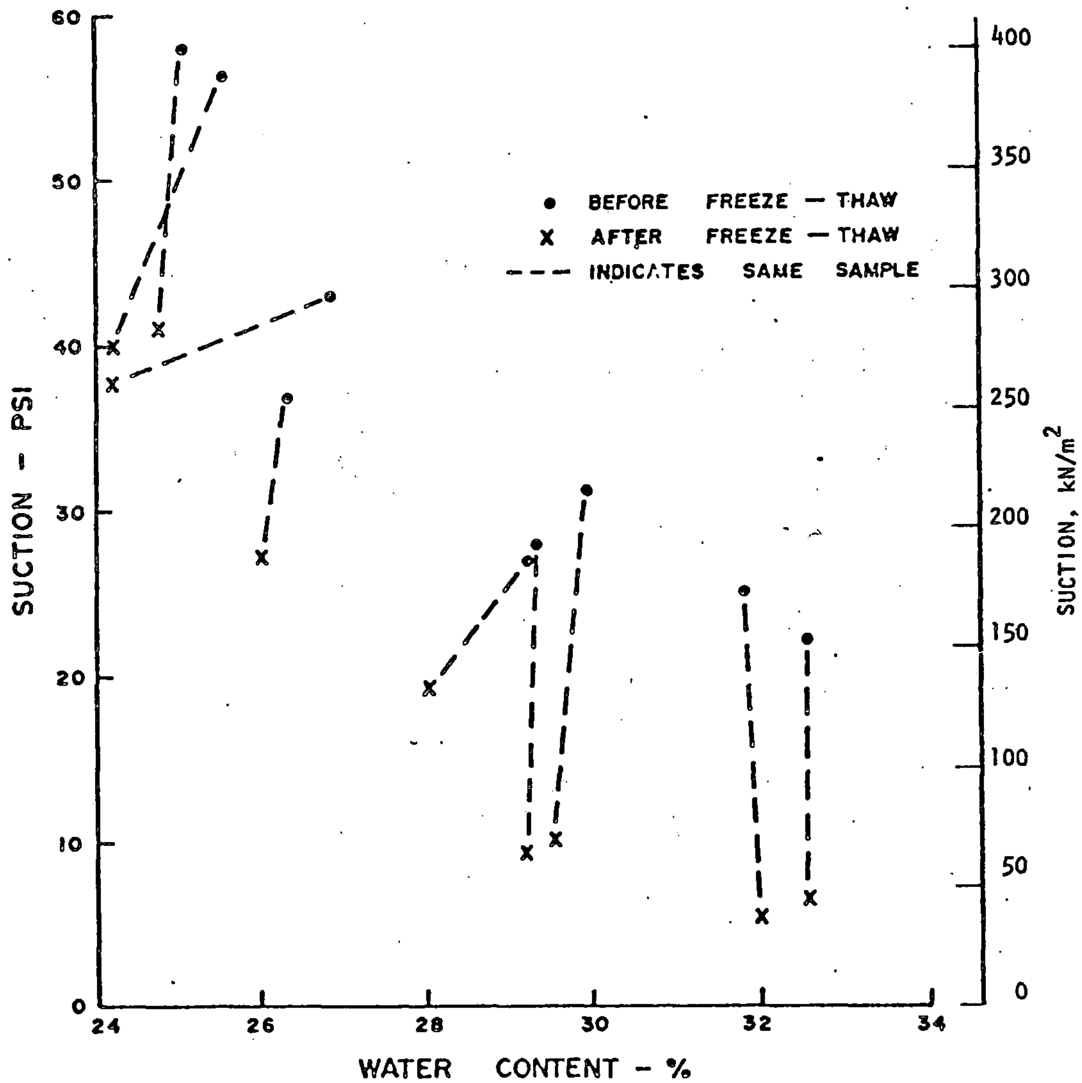


Figure 4.28. Effect of Freeze-Thaw Cycling on Soil Suction (From Ref. 4.23).

The repeated load resilient response of various fine-grained soils also can be substantially different, as shown by Figure 4.14.

CHAPTER 5

EXISTING METHODS OF ANALYSIS AND DESIGN OF RAILWAY SUPPORT SYSTEMS

GENERAL

Dating to the days of the building of the locomotive engine by George Stephenson in 1814, the design of the railway support system has been mostly empirical. In fact, in the early days railway support systems were not designed, but were rather constructed. To a large extent, this is also true today in the U. S. Over the years only sporadic efforts have been made in the United States to develop a better understanding of transient loading behavior of the railway support system. Extensive studies and advancements have been made in other countries (Great Britian, Germany, Japan, etc.). Recently, the need has been recognized in the U. S. for a continuing effort in developing a better understanding of the behavior and design of the railway track structure.

The primary purpose of this chapter is to examine currently available methods for the analysis and design of conventional railway support systems. Both theoretical and empirical methods are considered.

At the end of the chapter pertinent limitations of present methods and procedures are presented.

ANALYTICAL METHODS

Beam on Elastic Foundation Method

Significant contributions were made by Professor A. N. Talbot and the Special Committee to Report on Stresses in Railroad Track (5.1) in understanding the behavior of a railway track system subjected to loading. The concept of "track foundation modulus" was introduced and mathematical formulations were developed for calculation of the deflection and moment in the rail using the theory of a continuous beam on an elastic foundation as introduced by Zimmermann in 1888 (5.2). A brief outline of the mathematical formulation is given below:

For a beam loaded with a distributed load, q

$$EI \frac{d^4y}{dx^4} = q \tag{5.1}$$

For an unloaded beam on an elastic (Winkler type) foundation,

$$q = -ky \tag{5.2}$$

$$\therefore EI \frac{d^4y}{dx^4} = -ky \tag{5.3}$$

Using $\beta = \sqrt[4]{\frac{k}{4EI}}$, the solution for Equation 5.1 is given by

$$y = e^{\beta x}(A \cos \beta x + B \sin \beta x) + e^{-\beta x} (C \cos \beta x + D \sin \beta x) \tag{5.4}$$

For a point load on an infinitely long beam

$$y = e^{-\beta x} (C \cos \beta x + D \sin \beta x) \tag{5.5}$$

Solving for the constants, $C = D = \frac{P}{8\beta^3 EI}$

$$\therefore y = \frac{Pe^{-\beta x}}{8\beta^3 EI} (\cos \beta x + \sin \beta x) \tag{5.6}$$

and

$$M = -EI \frac{d^2y}{dx^2} = - \frac{Pe^{-\beta x}}{4\beta} (\sin \beta x - \cos \beta x) \tag{5.7}$$

$$\text{Max } y_{(at\ x\ =\ 0)} = \frac{P\beta}{2k} \text{ ; Max } M_{(at\ x\ =\ 0)} = \frac{P}{4\beta}$$

Symbol Description:

E, I = beam (rail) modulus of elasticity and moment of inertia

q = distribtued load on the beam

x = longitudinal coordinate of the beam

y = deflection of the beam

A,B,C,D = constants

P = point load acting on the beam at x = 0

M = moment in the beam

k = track foundation modulus

The "track foundation modulus" was defined as an elastic constant which denotes "the pressure per unit length of each rail necessary to depress the track (rail, tie, ballast, and subgrade) one unit." Theoretically, it represents the stiffness of tie, ballast and soil, but does not involve the stiffness of the rail. However, in actual field measurements of the track modulus the stiffness of the rails is also involved as the track modulus is obtained by measuring the deflection of the rail under an applied load.

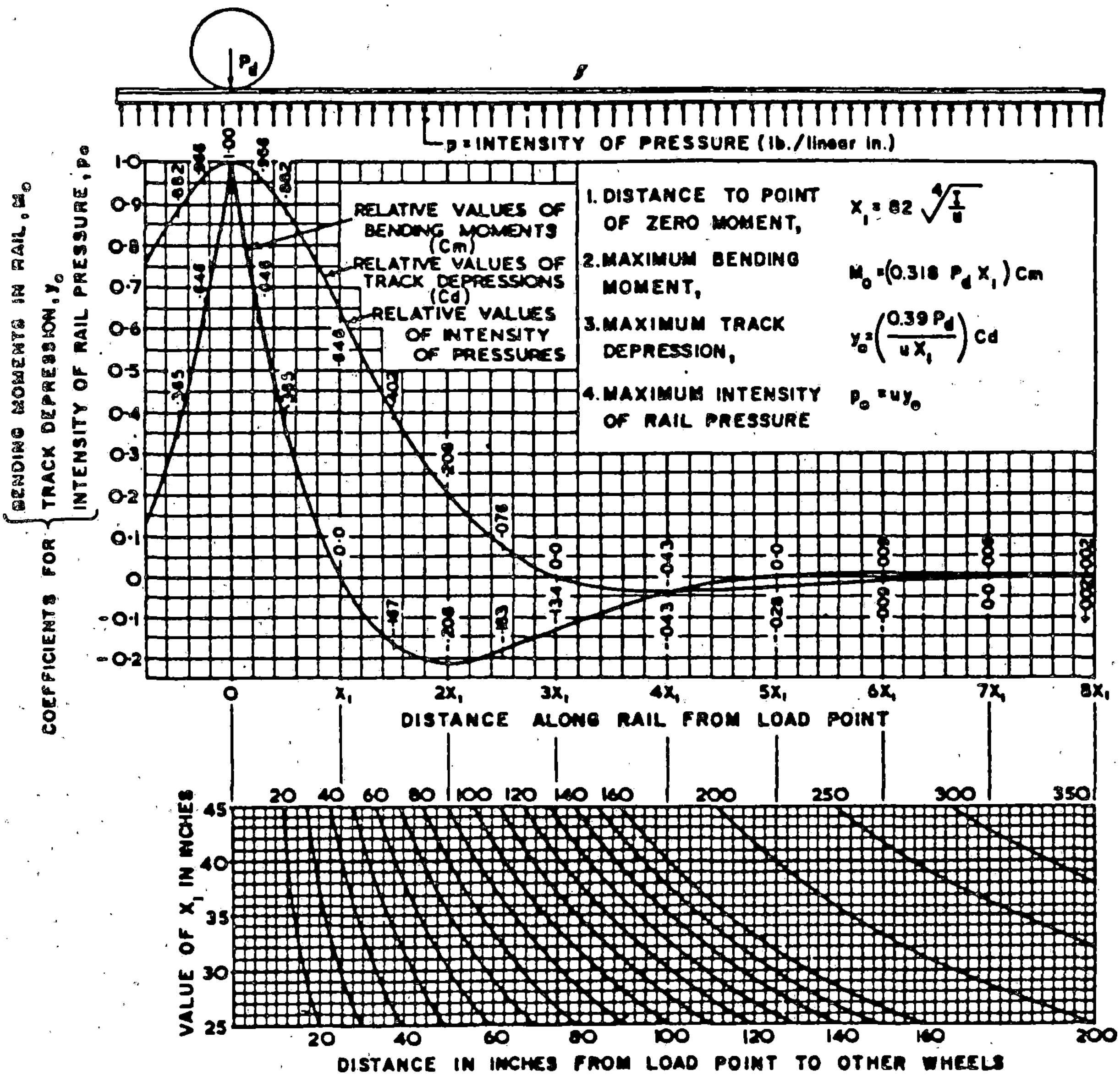
The solution of Equation 5.1 gives the moment, deflection and the shear stress at any point in the rail only. Rail deflection and rail bending moment can be determined using the curves shown in Figure 5.1. The effect of more than one axle is considered by the principle of superposition. Magee (5.3) has presented charts to evaluate tie reaction, tie depression, and rail bending stress for varying track parameters, using the beam on elastic foundation approach.

A major limitation to the beam on elastic foundation method is that it does not adequately model the mechanistic (stress-strain) behavior of the ballast and subgrade. Consequently the procedure is of limited value in considering the behavior of the sub-structure beneath the rail.

Meacham's Modification of Beam on Elastic Foundation Method

Meacham and his colleagues at the Batelle Memorial Institute tried to overcome some of the limitations of Talbot's model as given in Equation 5.3 by developing a theoretical approach for the evaluation of the track modulus value (5.4). The approach permits the use of track parameters such as rail type, tie type, ballast depth, ballast type, subgrade type, and tie-spacing as inputs in the analytical model. The essential steps involved in the development of the model are shown in Figure 5.2. An approximate method considering the affected volume of ballast to be a pyramid of uniform pressure is used. The angle of internal friction of the material determines the rate at which the load is assumed to spread out as it is transmitted downward, Figure 5.2a. Using the pyramid approximation, the ballast stiffness for a ballast depth, L, is given by

$$K_b = \frac{C(\ell-w)E_b}{\ell n\left[\frac{\ell}{w}\left(\frac{w+CL}{\ell+CL}\right)\right]} \qquad (5.8)$$



Design Steps

1. Calculate X_1 from formula 1 above.
2. Enter lower diagram with X_1. Draw horizontal line to intersect line indicating distance to adjacent wheels.
3. Draw vertical line to intersect upper graph curves giving bending moment and depression coefficients.
4. Read coefficients C_m and C_d on left scale.
5. Repeat for other adjacent wheel loads and enter totals in formulas 2 and 3.

Figure 5.1. Normalized Curves for Rail Deflection & Rail Bending Moment (From Ref. 5.14).

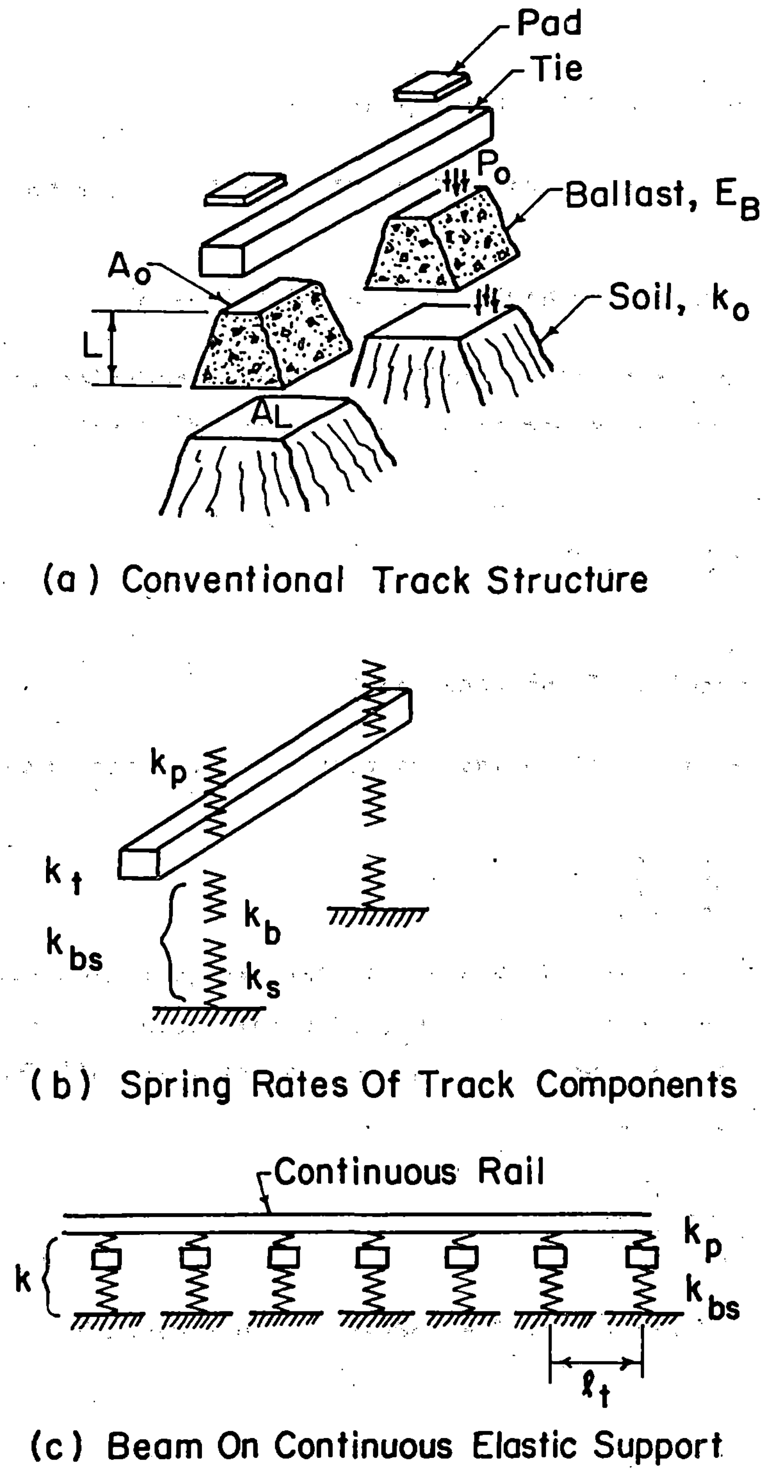


Figure 5.2. Progressive Steps in the Development of Static Model of a Conventional Track Structure (From Ref. 5.4).

where:

E_b = Young's modulus for ballast

ℓ, w = length and width of the effective bearing area of tie ($\ell > w$)

$C = 2 \tan \phi$ (ϕ = angle of internal friction)

The representation of the soil stiffness is modified to account for the bearing area A_L on the subgrade. Thus, the modified soil stiffness K_s is given by

$$K_s = K_o A_L = K_o \ (\ell + CL) \ (w + CL) \qquad (5.9)$$

where:

K_o = modulus of subgrade reaction.

The ballast-soil spring constant, K_{bs}, is then given by

$$\frac{1}{K_{bs}} = \frac{1}{K_b} + \frac{1}{K_s} \qquad (5.10)$$

The spring constant of each tie is the series equivalent of the spring constant of a resilient rail pad, K_p, plus half (due to continuity of ballast and soil) the spring constant of the ballast-soil foundation, K_{bs}, beneath the tie, i.e.,

$$\frac{1}{K} = \frac{1}{K_p} + \frac{1}{K_{bs}/2} \qquad (5.11)$$

The overall foundation modulus is then given by

$$k = \frac{K}{\ell_t} \qquad (5.12)$$

where:

ℓ_t = tie spacing

Numerical approaches for the analyses of a finite beam on an elastic (winkler) foundation have been presented by Barden (5.7) which allow non-uniform foundation modulus and by Harrison (5.8) which allow non-uniform beam section and non-uniform foundation modulus. Barden's representation is shown in Figure 5.3 and the discrete element representation used by Harrison for beam on elastic foundation is shown in Figure 5.4.

Analysis of Track Structure Subjected to Moving Loads (5.9)

Several theoretical models exist to determine the stress and deflection of the rail subjected to moving loads. The rail is treated as a beam resting on a continuous foundation. The foundation is idealized as one of the following:

1. A linear, isotropic, elastic solid.
2. A Winkler foundation in which the deflection at any point is assumed to be proportional to the load at that point, but independent of the load and deflection at any other point. It is physically represented by an infinite number of independent linearly elastic springs.
3. A one-dimensional linear viscoelastic element. Such a material incorporates damping and time effects. Two models of viscoelastic behavior that have been used are:

    a. Kelvin model, which is represented by a dashpot connected in parallel with a spring and

    b. Standard linear solid which consists of a spring placed in series with a Kelvin model.

The basic differential equation of motion for a beam supported by a flexible foundation may be written as follows:

$$EI \frac{d^4y}{dx^4} + \rho \frac{d^2y}{dt^2} = F(x,t) - q(x,t) \qquad (5.16)$$

where:

y = displacement of the point of the beam at time, t, and whose coordinate is x

The above value of the track modulus is then used in Equation 5.3. The pressure on the ballast surface is given by

$$P_b = \frac{Ky}{A_o} \qquad (5.$$

and the pressure on the subgrade is

$$P_s = \frac{P_b A_o}{A_L} \qquad (5.1$$

where:

A_o is the effective bearing area of the tie.

<u>Finite Beam on an Elastic Foundation</u>

The beam on an elastic foundation approach has also been used to analy the response of a tie resting on the ballast by considering the tie as a finite beam resting on an elastic (Winkler type) foundation. The governing differential equation is similar to Equation 5.1. The solution for the finite beam is obtained by first solving the problem of an infinitely long beam and then removing the moments and shearing forces at the ends of a finite length beam by superimposing reactions at those locations (5.5, 5.6). The formulations and the solutions are given in detail in References 5.5 and 5.6. This approach gives the moment and deflection of the tie (finite beam) due to the tie reactions. An equation for the ballast pressure, σ_z, under the tie is given below (Ref. 5.6):

$$\sigma_z = \frac{2k}{\pi b} w(x) (1-\eta^2)^{-1/2} \qquad (5.15)$$

where:

k = the effective foundation modulus and is a function of the elastic properties of the foundation (or ballast) as well as the beam (or tie) geometry.

b = width of the beam (or tie)

$w(x)$ = deflection of the beam (or tie) at distance x

$\eta = \frac{2y}{b}$

y = offset distance from the center line of the beam (or tie)

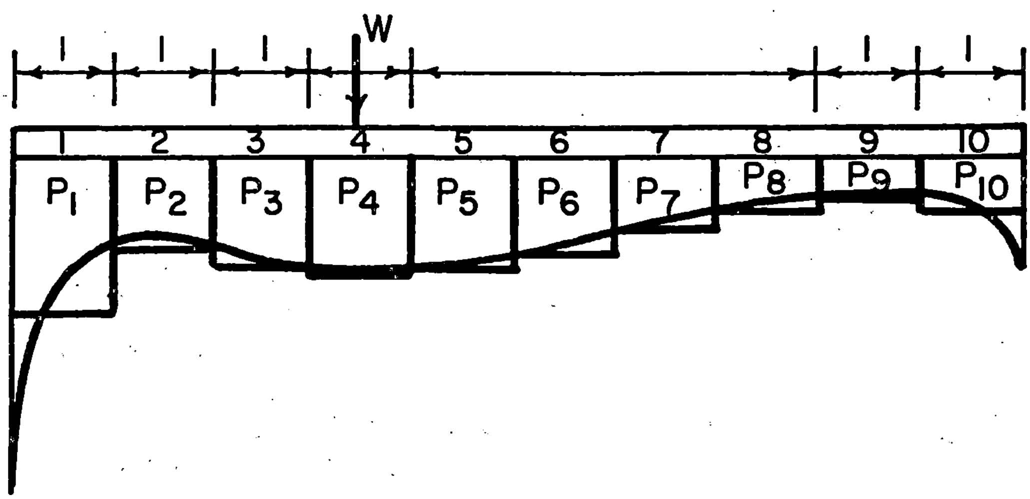


Figure 5.3. Finite Beam on an Elastic Foundation Model of Barden (From Ref. 5.7).

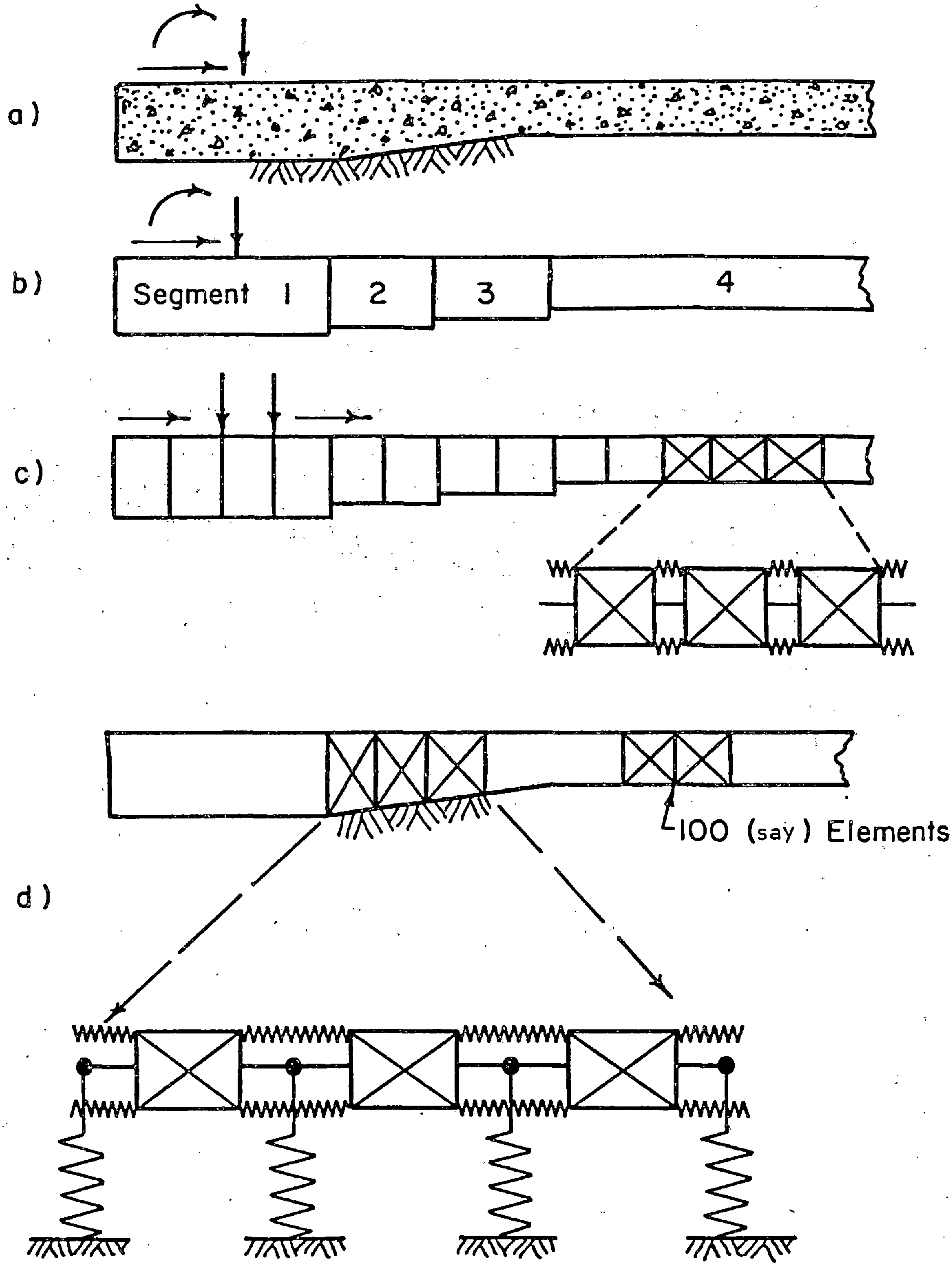


Figure 5.4. Discrete Element Representation of an Elastically Supported Beam used by Harrison (a) actual foundation beam, (b) idealization for data preparation, (c) final discrete element idealization by computer, (d) final discrete element idealization of beam on elastic foundation by computer (From Ref. 5.8).

E = modulus of elasticity of the beam

I = moment of inertia of the beam

ρ = density per unit length

$F(x,t)$ = impressed force, and

$q(x,t)$ = restoring force due to the foundation

Solutions of the above equation for a beam subjected to moving loads and resting on a linear, isotropic, elastic solid have not been developed as yet. Solution of the above equation for a beam subjected to various loading conditions and resting on the Winkler and linear viscoelastic foundations mentioned previously are available (5.9). Rostler et al. (5.9) state in their report that "there is no justification for the use of the viscoelastic models in quantitative stress analysis because of lack of experimental data for the parameters." After a review of available literature, it appears that situation still prevails.

Meacham, et al. (5.4) have reported that for the case where the rails are perfectly straight, the wheels perfectly smooth, and the track structure has perfectly uniform properties along its length and for speeds up to about 250 mph, there is very little difference between the static and dynamic vertical deflection due to the inertia and damping effects associated with speed. That is to say, a plot of deflection versus distance for static wheel loads would be approximately the same as a plot of deflection versus time taken at any point on the track structure for conventional train speeds.

General Boussinesq Approach

An approximate analytical method exists in which the wheel load is assumed to be distributed over a certain number of ties in a certain proportion. The bearing area of the ties is assumed and the vertical stress distribution with depth under any given tie is found using the Boussinesq theory as given by Newmark's graphical solutions (5.10). The stress distribution due to all the effective ties is then found by superposition. Ireland (5.11) has discussed such an approach in an address to an AREA Regional Meeting and has presented a design chart for ballast-subballast depth selection versus cohesive strength of subgrade soil. The British Railways Design method (subsequent discussion) uses a similar approach to determine the stress distribution with depth.

Finite Element Models

A finite element model developed by Lundgren, Martin and Hay (5.12) at the University of Illinois represents the track structure as a two-dimensional system and considers the behavior of a longitudinal section of unit thickness along the vertical centerline of the rail. The condition of plane strain is assumed valid for the section. The model uses 4 in. (10.16 cm) square elements with the rail-tie represented as a continuous beam on equivalent tie springs. The model allows calculation of stresses and deflections at various locations along the longitudinal section of the structural system.

The development of a three dimensional finite element model has been reported by the Canadian Institute of Guided Ground Transport (5.13). Details of the model are not as yet available. A three dimensional finite element model is also under development by the Association of American Railroads.

Stress Distribution with Depth (Semi-Empirical Methods)

a. Talbot Equations

Professor Talbot and his Committee to Report on Stress in Railroad Track (5.1) developed an equation which gives the pressure at different depths under the rail below the center line of the bearing area of a loaded tie. The equation is:

$$P_c = \frac{16.8\ P_a}{h^{1.25}} \qquad (5.17)$$

where:

P_c = the pressure directly below a loaded tie at depth h, inches

and P_a = the average pressure under the tie, psi.

The Committee also developed an empirical equation for the distribution of pressure on a horizontal plane. The equation for an 8 in. (20.3 cm) tie is

$$P'_c = \frac{16.8\ P_a}{h^{1.25}}\ (10)^K \qquad (5.18)$$

where:

P'_c = the pressure at a point on a horizontal plane h inches below a loaded tie and x inches from the center line of the tie, psi

and $K = -6.05 \dfrac{x^2}{h^{2.5}}$

The above equations reasonably agree with observed field results except for depths less than 4 in. (10 cm) and more than 30 in. (76 cm).

Other semi-empirical equations for calculating subgrade stress have been developed and are given below:

b. Japanese National Railways Equation (5.14)*

$$P_f = \frac{50\ P_a}{10 + D^{1.35}} \qquad (5.19)$$

where

P_f = the pressure on the formation

D = depth of ballast in centimeters

c. Deutsche Bundesbahn Equation (5.15)*

$$P_f = \frac{1.5\ (L-S)\ B\ P_a}{[3\ (L-S) + B]\ D \tan\theta} \qquad (5.20)$$

where:

L = length of tie, B = width of tie

S = distance between center-lines of rails

θ = angle of dispersal of load

d. Indian Equation (5.15)*

$$P_f = \frac{2B}{\pi D} P_a \qquad (5.21)$$

e. Love's Equation (5.14)*

$$P_f = P_a \left[1 - \left(\frac{1}{1 + \frac{r^2}{h^2}} \right)^{3/2} \right] \qquad (5.22)$$

*original references could not be found.

where:

r = radius of a uniformly loaded circle whose area equals the effective tie bearing area under one rail seat.

DESIGN METHODS

The British Railway Design Method

British Railway Engineers have developed a design procedure which is based on "elastic" behavior and attempts to protect against excessive plastic deformation of the subgrade. Apparently it is applicable to design of systems with continuous welded rails (5.16, 5.17, 5.18, 5.19). The main assumptions inherent in the design procedure are:

1. Allowable threshold stress parameters for the subgrade soil may be obtained using standard repeated loading test (triaxial test),
2. Simple elastic theory (rectangular loading on semi-infinite linearly elastic foundation) can be used to predict the stresses in the subgrade from traffic loading,
3. The significant traffic stresses are those produced only by the static effect of the heaviest commonly occurring axle load, and
4. The water table is at the top of the subgrade.

The "threshold stress" for a clayey soil is defined as the stress level above which the deformation of the soil under repeated load is very rapid; below the threshold stress level the rate of cumulative deformation from additional repeated loading is very small.

The design procedure involves essentially two stages, namely, a stress analysis stage and a stage in which ballast thickness design charts are developed.

a. Stress Analysis Stage

A transverse stress analysis (along the length of a tie) is conducted. The interface pressure distribution under the tie is first evaluated using a procedure similar to that presented by Barden (5.7), in which a beam on an elastic foundation is divided into a finite number of segments.

Boussinesq approach is then used to calculate the stresses in the interior of the track foundation (which is considered as a semi-infinite elastic solid). The effect of several ties is obtained by superposition.

b. Design Chart Development Stage

Design charts such as shown in Figure 5.5 are prepared by considering the stress distribution in the system due to various axle loads and design speeds and the "strength" of the subgrade soil.

The "strength" characteristics of the subgrade soil are determined using a repeated load triaxial test. In general, the design method provides a ballast thickness such that the shear stresses as calculated by the above procedure are less than the "strength" or "threshold stress" of the subgrade.

AREA Design Guidelines (5.20)

In these guidelines, the thickness of ballast is estimated by considering the strength of the subgrade soil based on laboratory testing of saturated, remolded samples (static triaxial test) and by considering the stress-distribution in the foundation. A discussion of this approach is given by Ireland in Ref. 5.11. The AREA design guidelines also suggest a minimum of 12 in. (15 cm) of top ballast under ties and 6 in. (30 cm) of sub-ballast.

Summary of Current Design Methods Used by U.S. Railroads

Many of the major U. S. railroads have standard track designs for use in specific regions of their areas of operation. Most of these designs have been developed by experience rather than theory. For example, the Atchinson, Topeka, and Santa Fe specifications for main track construction in the region of Kansas Test Track (45 miles northeast of Wichita) call for (5.21):

1. use of continuously welded 136 lb./yd. (68 kg/m) rail with 1440 foot (438.9 m) rail string ends field welded;
2. use of 7 in. by 8 in. by 9 ft (17.8 cm by 20.3 cm by 2.74 m) hardwood, treated cross-ties spaced at 19.5 in. (49.4 cm) centers;
3. a minimum of 10 in. (25.4 cm) of crushed, ferrous metal slag ballast beneath the bottom of ties. About 6 in. (15.2 cm) of lime stabilized subgrade is also used.

Many other railroads, depending on traffic volume and experience use anywhere from 6 to 12 in. (15.2 to 30.5 cm) of ballast and from 4 to 12 in. (10.2 to 30.5 cm) of sub-ballast. Thicker ballast sections may be present

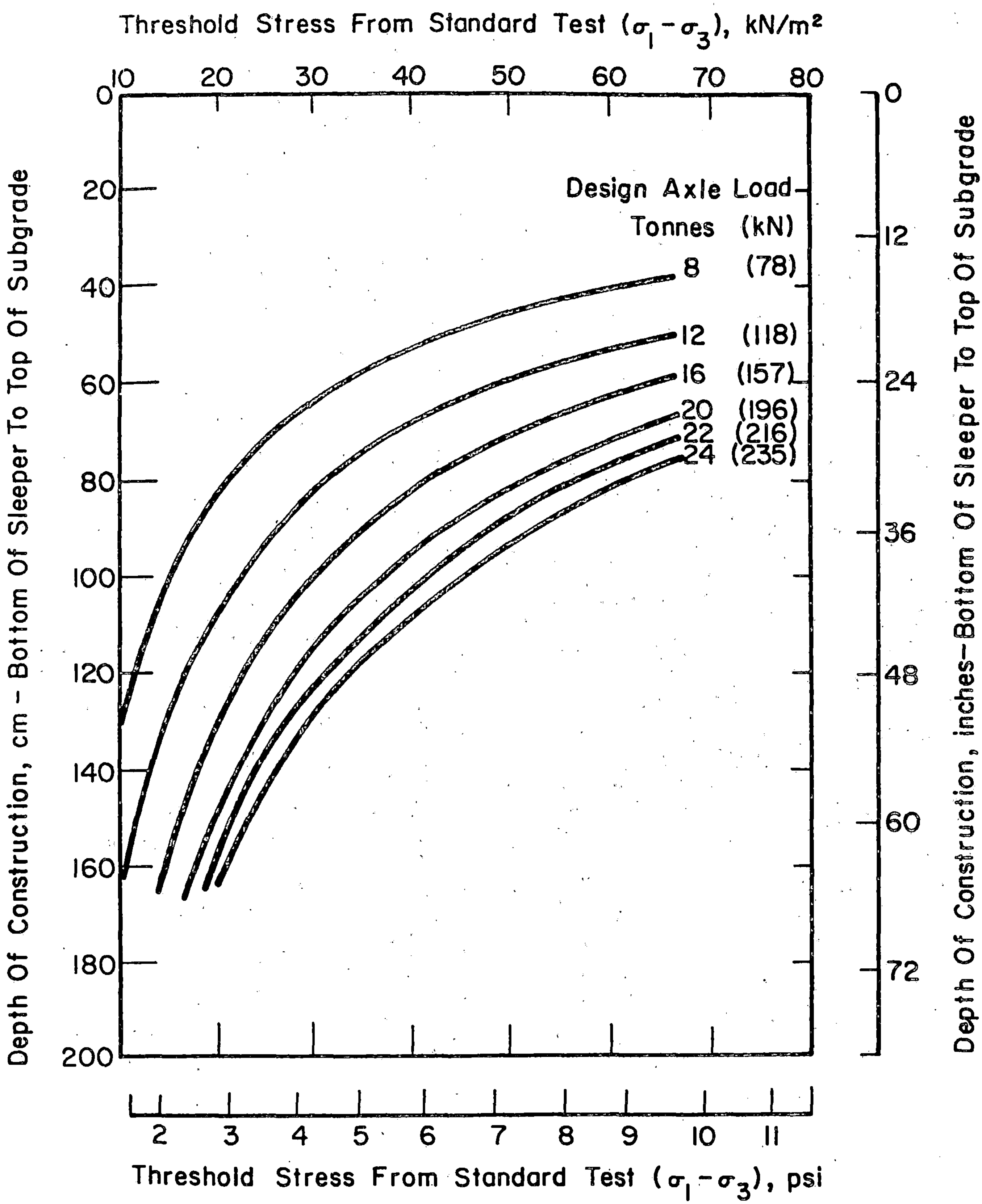


Figure 5.5. The British Railways Design Chart (From Ref. 5.16).

under some tracks as a result of periodic maintenance operations (surfacing, track raising, etc.). It is interesting to note that some of the European railroads use substantially thicker ballast and subballast sections even through their vehicle loads in general are less than those in the United States.

SUMMARY AND CONCLUSIONS CONCERNING EXISTING METHODS OF ANALYSIS AND DESIGN

Numerous theories, techniques and/or procedures have been developed for calculating stresses and deflections or deformations in the railway support system. Most however have concentrated on "realistic" representation of the rail-fastener-tie components while representing the ballast and subgrade as either springs or linear elastic, homogeneous materials.

Based mainly on experience and in some cases on available theory, a number of design procedures have been developed for "designing" the thickness of ballast required in the track support structure. It is significant to note however, that with the exception of the British Railway procedure, none of the "design" procedures incorporates the repeated load strength characteristics of the ballast and subgrade materials. Some of the design procedures incorporate "static" subgrade strength characteristics but none consider the effects of ballast properties or characteristics on ballast thickness design.

Additionally, other important design considerations such as climatic or environmental exposure and traffic volume are not considered.

CHAPTER 6

EFFECT OF BALLAST AND ROADBED MATERIAL STRENGTH PROPERTIES ON TRACK SUPPORT SYSTEM RESPONSE

GENERAL

The major factors that affect the structural behavior of a conventional track support system subjected to loading are:

1. rail properties (size, joints, etc.)
2. tie spacing
3. ballast and subballast properties (strength and thickness) and
4. roadbed or subgrade properties

The "Ballast and Foundation Materials Research Program" as the name implies is primarily concerned with ballast and foundation or roadbed materials. The track support system behavior of primary concern is the "elastic" and "plastic" response to in-service loading conditions. The objective of this chapter is to briefly discuss the general sensitivity of track support system behavior to ballast and roadbed material properties.

As a general guide, reduced levels of track support contribute to a faster rate of track deterioration. However, a rational railway support system design approach requires more substantiative answers concerning the effect of support system material properties on track response.

PREVIOUS RESEARCH FINDINGS

A Canadian Field Study reported by Hardy (6.1) indicated that the thickness of loosely placed ballast had little influence on rail stresses and maximum subgrade deflection. The ballast materials used in the study were loosely placed crushed gravel and loosely placed pit-run gravel. Ballast depths were 8, 18, and 30 in. (20.3, 45.7, and 76.2 cm).

Very little has been reported in recent literature about actual field studies in which the ballast and the subgrade properties were the major variables investigated. Kamath and Mukerjee (6.2) reported on static load tests conducted by the Research Designs and Standards Organization, Lucknow, India in which the main variables were a) sleeper type and spacing, b) ballast depth and packing condition, and c) use or non-use of blanketing materials of different depths. Their test results indicated that about 20 to 25 percent reduction in maximum intensity of pressure on the roadbed was obtained by increasing the ballast thickness from 8 in. to 10 in (20.3 cm to 25.4 cm).

The Office of Research and Experiments (ORE) (6.3) reported that the means and standard deviations of settlement and track deflection increase when the depth of ballast increases. Thus, for thicker ballast sections the settlement at different positions along the track is less uniform and tends to accentuate track surface defects.

Limited data are available to evaluate the response of different types of ballast. Salem (6.4) investigated the behavior of three types of ballast - chat, pit-run gravel, and crushed slag. As can be seen from Figure 6.1, the vertical pressure distribution with depth was similar for the three ballast materials.

The track resistance to transverse and longitudinal displacements on the ballast has been found to depend on both the ballast compactness and the size of ballast particles. Birman's (6.5) study indicates the resistance of a track panel on new wooden sleepers subjected to lateral displacements on firmly tamped ballast of size 0.6-1.2 in (15-30 mm) was about 30 percent less than the lateral resistance of the same track panel on conventional, coarse ballast of size 1.2-2.4 in. (30-50 mm).

The strength properties of ballast are highly dependent on the type of "consolidation" process used to prepare or to maintain the ballast bed as well as the density of the ballast. This is especially true for the case of lateral stability of the track. Measurements at the Technical University, in Munich (6.6) showed that the lateral resistance was greatest after the track had settled under traffic. After tamping and a track raise of about 0.10 in (2 to 3 mm), the lateral resistance decreased by approximately 70 percent as shown in Figure 6.2. The effect of tamping was shown to be partly offset by using track "compactors" (vibratory compacting machines) which follow closely behind the tamping machine. It was also shown that after tamping the vertical track stiffness is reduced. The use of vibratory equipment to compact the ballast in cribs between ties has been suggested

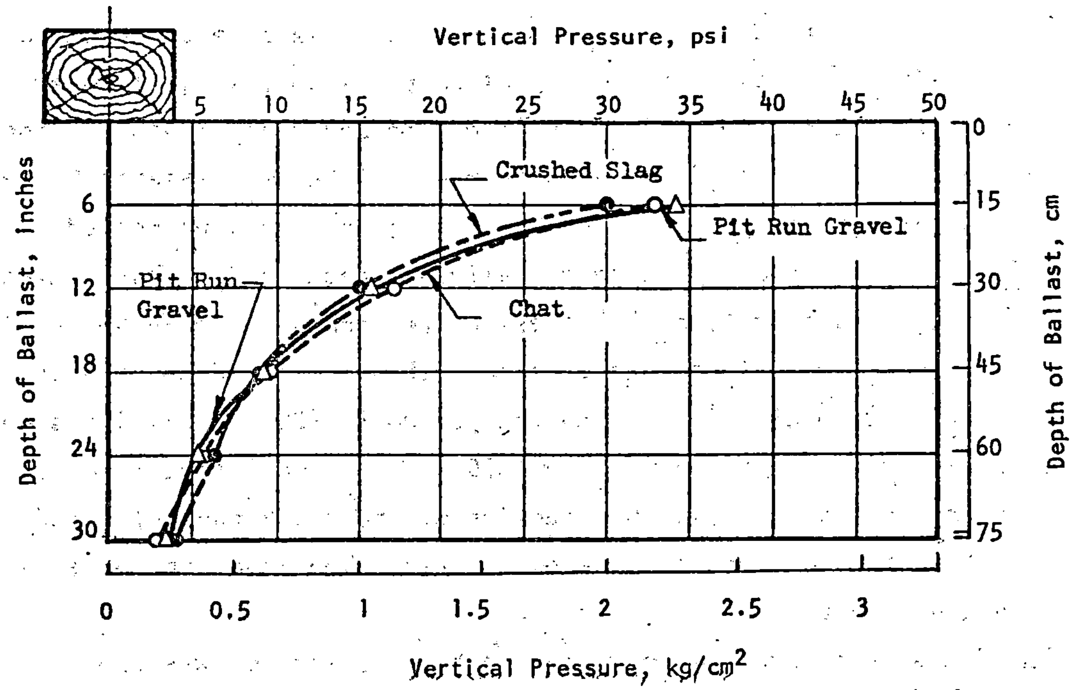
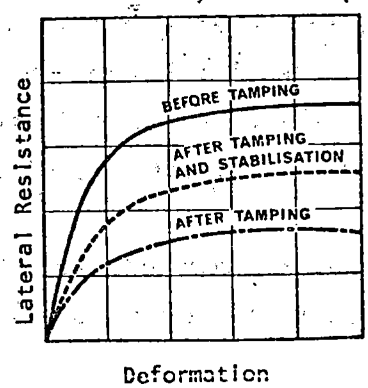


Figure 6.1. Vertical Pressure Distribution at Depths up to 30 in. of Ballast Below the Center-Line of a Single Tie, Tie Load = 20,000 lb. (From Ref. 6.4).



Note; No scale given in original reference.

Figure 6.2. Relationships Between Lateral Resistance and Deformation Obtained from the Tests Carried Out on a Length of Track Comprising of Seven Sleepers (From Ref. 6.6).

by Klugar (6.7). Klugar indicated that ballast compaction effected increased horizontal confinement in the ballast material underneath the ties.

In a pilot study, Materials Research and Development, Inc. of Oakland, California (6.8) evaluated several stabilizing agents as to their potential for improving the properties of ballast materials. Treated and untreated ballast were evaluated by a) static triaxial compression tests, b) dynamic triaxial compression tests, and c) water immersion and freeze-thaw cycling tests. The results of the pilot study were used by the AAR Research Center to help plan an investigation of the behavior of treated and untreated ballast in a short full-scale section of railroad track (6.9). Twelve inches (30.5 cm) of crushed quartzite rock ballast corresponding to AREA No. 4 gradation was used. The ballast was treated with a new butadiene-styrene block copolymer based emulsion. In general 50,000 lb (22,680 kg) loads were applied either statically or dynamically (500 cpm) on two 6-foot (1.83 m) long sections of railroad track. One section of the track had untreated ballast and the other section had treated ballast. The important observations of this investigation are summarized below:

1. The settlement for the untreated ballast section was about 10 times the settlement for the treated ballast section after 3,000,000 load applications.
2. The approximate elastic settlements of the subgrade between ties were 0.02 in. (0.05 cm) under static loading, 0.01 in. (0.025 cm) under cyclic loading for the untreated ballast section; and 0.01 in. (0.025 cm) under both loading conditions for the treated ballast section.
3. The average subgrade pressure in the treated ballast section was about the same as the subgrade pressure in the untreated ballast section.
4. The accelerations of the ties and subgrade under cyclic loading were smaller for the treated ballast section than those for the untreated ballast section.
5. The lateral resistance of the unloaded track was about 5 times greater for the treated ballast section than the untreated ballast section.

Thus it is evident, for the AAR tests, ballast stabilization using the elastomeric polymers significantly improved the strength properties of the ballast.

During 1959 and 1960, several railroads in cooperation with the Research Department, AAR, the AREA Roadway and Ballast Committee, and The Asphalt Institute installed test sections of asphalt treated ballast (6.10). In each case an untreated section of equal length was marked off as a control section for comparison. The asphalt cement used was 85-100 and 120-150 penetration grade and was generally applied at a rate of about 1 gallon per square yard (4.50 liter/m^2) for the more densely graded ballast to about 2 gallons per square yard (9.0 liter/m^2) for very open and porous ballast. After a condition survey was conducted in 1962, it was reported that most of the test sections were in fair to excellent condition. It should be pointed out however that the primary reason for use of the asphalt treatment was to seal off the ballast surface from dirt and water intrusion.

For subgrade soils, repeated load behavior is very much dependent on the moisture condition. The effect of cyclic loading on the elastic and permanent deformation nature of cohesive soils has been discussed previously. During wet weather, ballast pocket formation is accelerated (6.11).

No methods presently exist to consider the stochastic variation in a track structure. Stochastic variations can be conveniently divided into three basic types as follows (6.12):

1. Variability within a project - such factors as ballast gradation, thickness, subgrade moisture content, density and strength.
2. Variability between assumed average design values and those obtained "as constructed" or to which the track structure is actually subjected during its design life.
3. Variability due to the inadequacy of the design procedure to account for all necessary factors and/or adjust for each factor in the correct manner.

A method of structural response analysis incorporating stochastic variations has been suggested by Darter, et al. (6.13) for highway pavement design. The use of discrete and finite element models to evaluate the effects of material variability on highway pavement response has also been suggested (6.14, 6.15). It is anticipated that similar approaches could be applied to the analysis and design of track structures.

THEORETICAL EVALUATION OF EFFECT OF BALLAST AND FOUNDATION MATERIAL PROPERTIES

Using the track modulus as representing the overall strength properties of the track support system, a theoretical analysis can be made using the Beam on Elastic Foundation method (Chapter 5). Figures 6.3 and 6.4 show the effect of track modulus on rail bending stress and deflection respectively for various rail sizes. For track modulus values below about 2000 lb/in/in (140 kgf/cm/cm) the rate of decrease in rail bending stress and deflection is higher than that for track modulus values above 2000 lb/in/in (140 kgf/cm/cm). It can thus be concluded, based on the beam on elastic foundation model data, that for a given system of loading, increasing the track stiffness beyond a certain limit yield increasingly less benefits.

A similar analysis can be made using Meacham's procedure (6.17) to determine the effects of ballast stiffness (in terms of E_B value), ballast depth, and subgrade modulus (in terms of K_{sub}). Table 6.1 shows the effect of ballast modulus, E_B and ballast depth on the overall track modulus, rail deflection, rail bending stress, tie-ballast bearing pressure, and ballast-subgrade pressure.

Lundgren, et al. (6.18) used their finite element model to evaluate the effects of ballast modulus, ballast thickness and subgrade modulus. Figures 6.5, 6.6, and 6.7 show the effect of ballast modulus, ballast thickness and subgrade modulus respectively on maximum track deflection and maximum rail moment. Higher ballast moduli result in decreased maximum rail movement and decreased maximum track deflection. For a given ballast modulus, an increase in ballast thickness generally results in an increase in maximum rail moment and a decrease in maximum track deflection. When the subgrade modulus is the variable, the results obtained were erratic as shown in Figure 6.7.

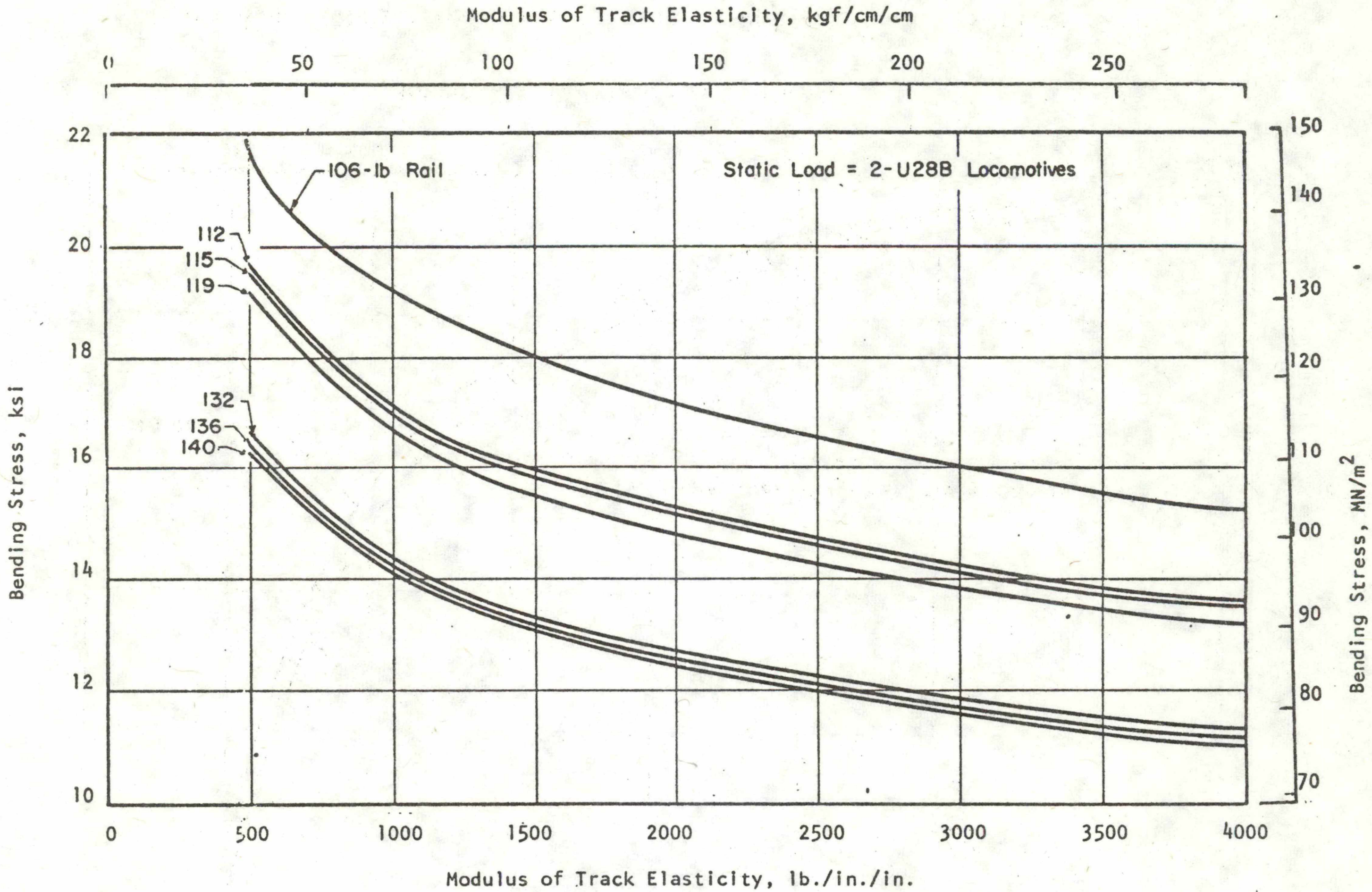


Figure 6.3. Effect of Track Modulus on Bending Stress for Various Rail Sizes (From Ref. 6.16).

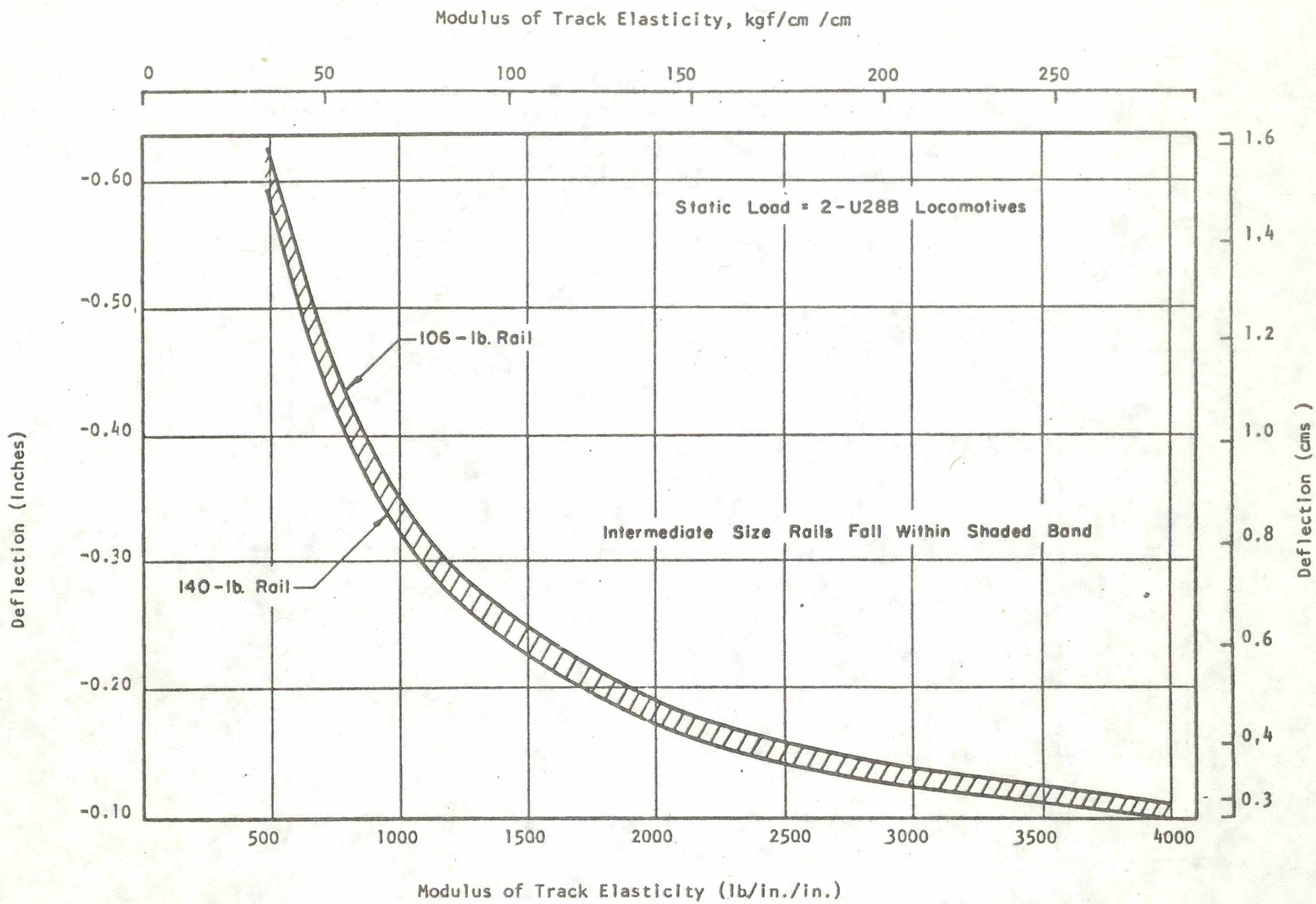


Figure 6.4. Effect of Track Modulus on Deflection for Various Rail Sizes (From Ref. 6.16).

Table 6.1. Effect of Material Property Variation on Track Structure Response (From Ref. 6.17).

Ties			Ballast/Subgrade			Rail		Ballast/Subgrade	
Type	Spacing, inches	Bearing Area, in.^{2}	Ballast Modulus E_B, lb/in.^{2}	Ballast Depth, inches	Overall Foundation Modulus, psi	Deflection, inches	Bending Stress, psi	Tie-Ballast Bearing Pressure, psi	Ballast-Subgrade Pressure, psi
Wood	18	2(229.5)	40,000	24	2792	0.172	10,450	37.6	7.6
Wood	18	2(229.5)	40,000	12	1609	0.284	12,100	35.9	13.6
Wood	18	2(229.5)	20,000	24	2500	0.190	10,750	37.2	7.5
Wood	18	2(229.5)	20,000	12	1538	0.296	12,250	35.7	13.5
MR-3	30	2(396)	40,000	24	2223	0.211	11,100	35.5	9.5
MR-3	30	2(396)	40,000	12	1381	0.327	12,600	34.3	15.7
MR-3	30	2(396)	20,000	24	2015	0.231	11,400	35.2	9.4
MR-3	30	2(396)	20,000	12	1325	0.340	12,700	34.2	15.6

*Two 35,000 # Wheel Loads at 6' Truck Axle Spacing, 136#/yd Rail, Subgrade Modulus = 100 lb/in.^{3}

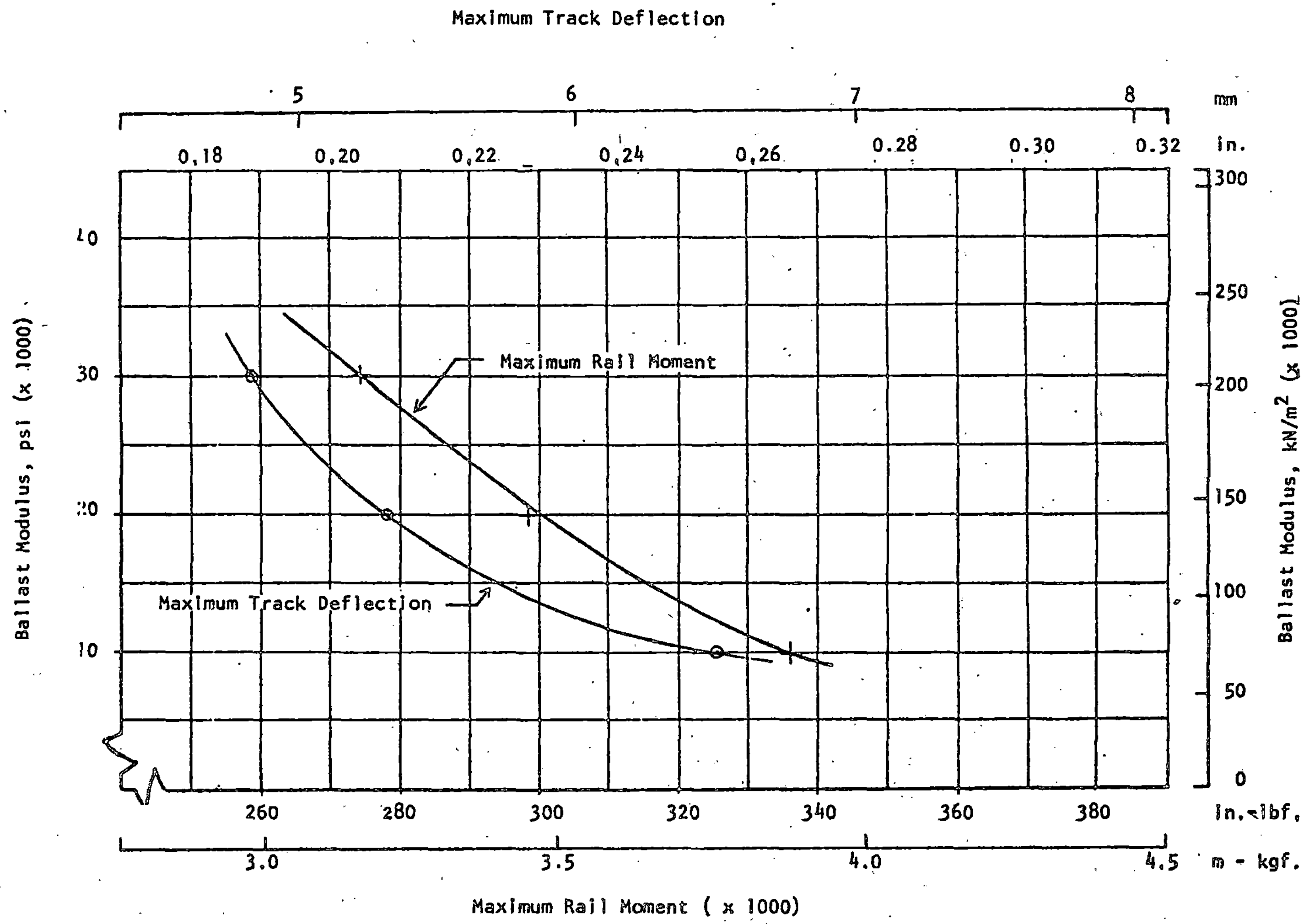


Figure 6.5. The Effect of Ballast Modulus on Maximum Rail Moment and Maximum Track Deflection (Ballast Depth = 24 in. (61 cm), Subgrade Modulus = 1000 psi (6895 KN/m^2)).

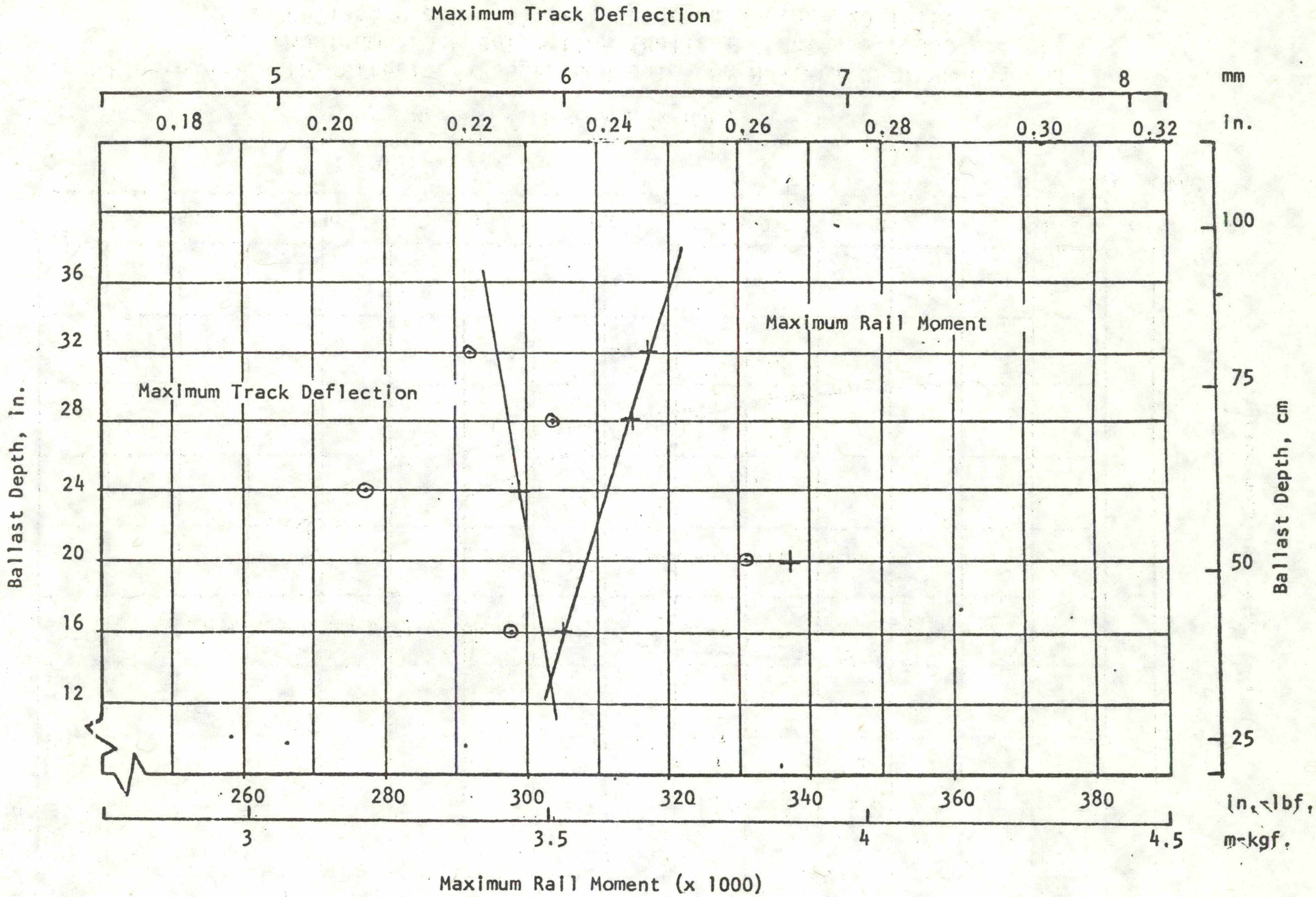


Figure 6.6. The Effect of Ballast Depth on Maximum Rail Moment and Maximum Track Deflection (Ballast Modulus = 20000 psi).

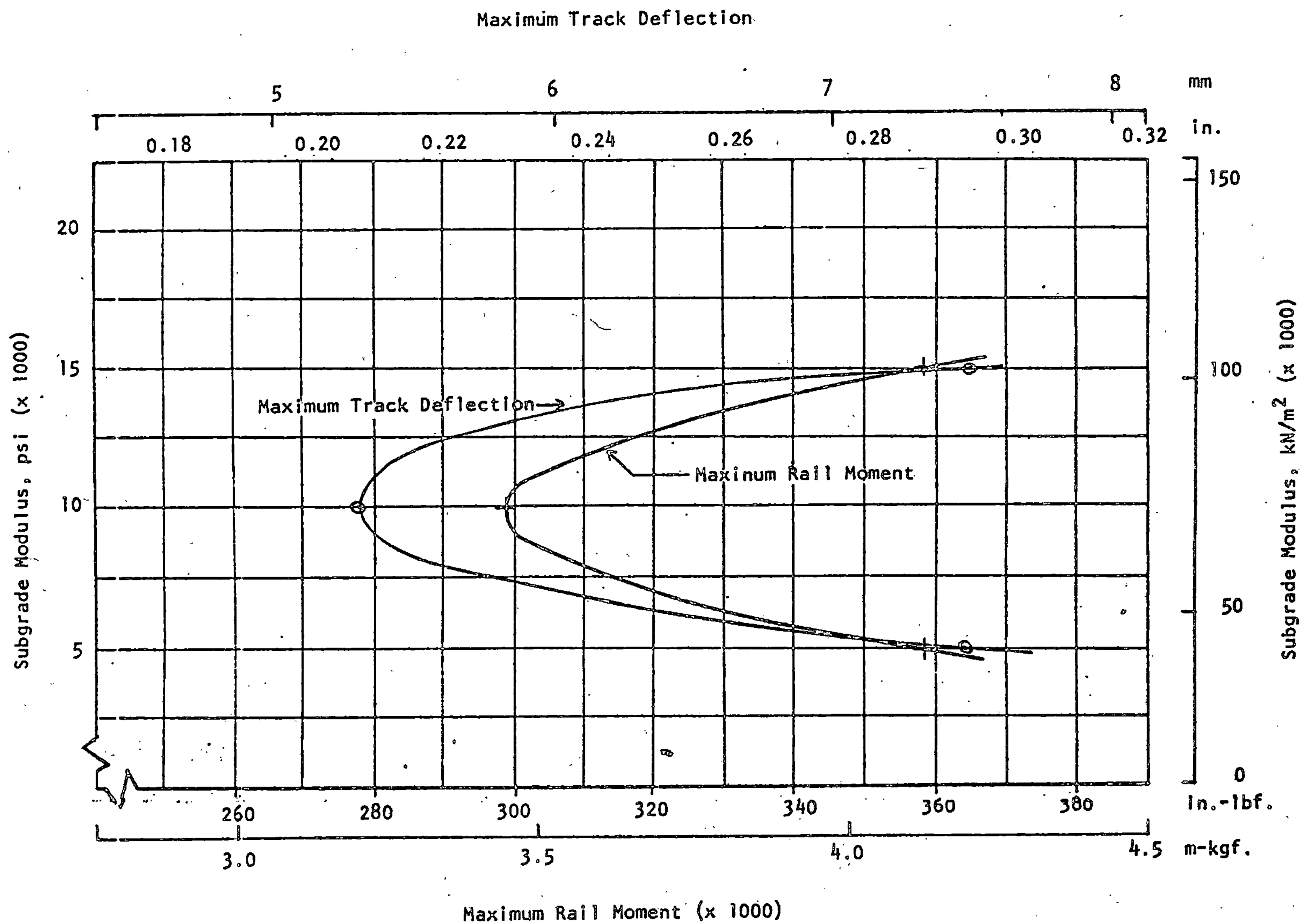


Figure 6.7. The Effect of Subgrade Modulus on Maximum Rail Moment and Maximum Track Deflection (Ballast Modulus = 20000 psi, Ballast Depth = 24 in., Subgrade Depth = 20 in.).

CHAPTER 7

CONSIDERATIONS IN PREDICTING PERFORMANCE OF RAILWAY SUPPORT SYSTEMS

GENERAL

An integral part of rational railway support system design is to be able to predict the performance of the system. This means that the designer must be able to quantify the significance of the design he has developed. He must be able to answer questions such as:

1. For the projected traffic, how long will the system last before "maintenance" of some sort is required?
2. What effect will changing certain properties of the support system (for example ballast thickness, ballast type, tie spacing, rail characteristics, etc.) have upon the "time" to first maintenance?
3. What are the economics associated with various designs and various maintenance strategies?

PERFORMANCE DEFINED

In order to discuss the state-of-the-art for predicting railway support system performance it is first necessary to define the concept of performance and point out some of the factors of influence.

The word "performance" may be somewhat nebulous but in general it is defined by Webster as "operation or functioning, usually with regard to effectiveness...".

In the highway field, performance is defined as the adequacy with which a pavement fulfills its intended purpose or function. Implicit in this general definition is the time factor. The general function or purpose of a highway pavement is "to provide the surface conditions necessary for the safe, comfortable, economical operation of vehicles using the pavement."

It seems reasonable to extend this general highway definition of performance to the railway system. The function or purpose of a railway support system is probably also similar to the highway definition, i.e. "to provide the rail surface conditions necessary for the safe. comfortable, economical operation of trains using the system."

In the highway field, a numerical rating index called the Present Serviceability Index (7.25) has been developed in which the relative ability of the pavement to perform its intended purpose or function can be obtained by measuring certain pavement surface properties.

Based on the railway literature it appears that equipment and methodology have been developed to evaluate certain conditions of the rail system. Such properties as cross level, longitudinal roughness, longitudinal differential settlement, etc. are currently evaluated with various rail inspection cars and other procedures. It is obvious that the aforementioned track conditions closely relate to general safety, comfort, and economy. For example, if intolerable levels of certain of these track conditions exist, then freight damage, derailments, equipment damage, etc. may occur.

It should be acknolwedged, that the tolerable levels of certain pertinent track surface conditions will be different for various users. For example, a high speed passenger train requires different levels of track surface conditions than does a typical freight train.

It should also be pointed out that the "users" (i.e. the trains and crews) are not concerned with the structural design aspects of the support system, but rather the surface conditions displayed by the support system.

The main objective of the designer is to provide a structural support system that will exhibit tolerable levels of surface conditions for a predetermined period of time or number of load applications. In order to do this, it is necessary to have relations between structural response and the trend of surface condition deterioration as a function of time and/or load applications.

TRANSFER FUNCTION CONCEPT

In general, the transfer function concept is that there exists various relations between structural response and "performance" or distress development of the structural support system.

By definition, a transfer function is the ratio of an operational output to an operational input of a time-dependent system (7.1). For a movement support system, a transfer function is considered to be a relationship that relates the functional response (structural response) of the movement support system to its performance. A well known example of this relationship is the result of the repeated load flexural test on concrete beams. The number of load applications to failure is dependent on the stress level in the beam.

Current Status of Transfer Functions for Highway Pavement Design

Considerable progress has been made in developing transfer functions for use in design and analysis of highway and airfield pavements. In general, critical structural response features have been identified and performance is predicted using transfer functions based on these critical response features. Transfer functions which are currently used in the design of highway and airfield pavements and which appear to be relevant to design of the railway support system are discussed below.

a. Surface Deflection

It is generally accepted that the greater the pavement surface deflection under traffic the shorter is the pavement's service life. In a study in Britain (7.2) well defined relationships were obtained between critical lives of flexible pavements and mean early life deflections. For lives (N load applications) of greater than about one million standard axles, the curves were of the form:

$$\text{Life (N)} \propto \frac{1}{(\text{Deflection})^3} \tag{7.1}$$

A typical early deflection - life curve is shown in Figure 7.1. Similar results were also obtained during the AASHO road test (7.3) and equations of the form shown below were developed to predict performance of flexible pavements.

$$\log w_p = A_o + A_1 \text{ Log } L_1 - A_2 \log d \tag{7.2}$$

where:

w_p = number of applications of axle load L_1 sustained by the pavement at the time the serviceability was at level p

L_1 = single axle load, in kips;

d = normal deflection measured under a wheel load equal to one half L_1, in 0.001 in.; and

A_0, A_1, A_2 = regression coefficients.

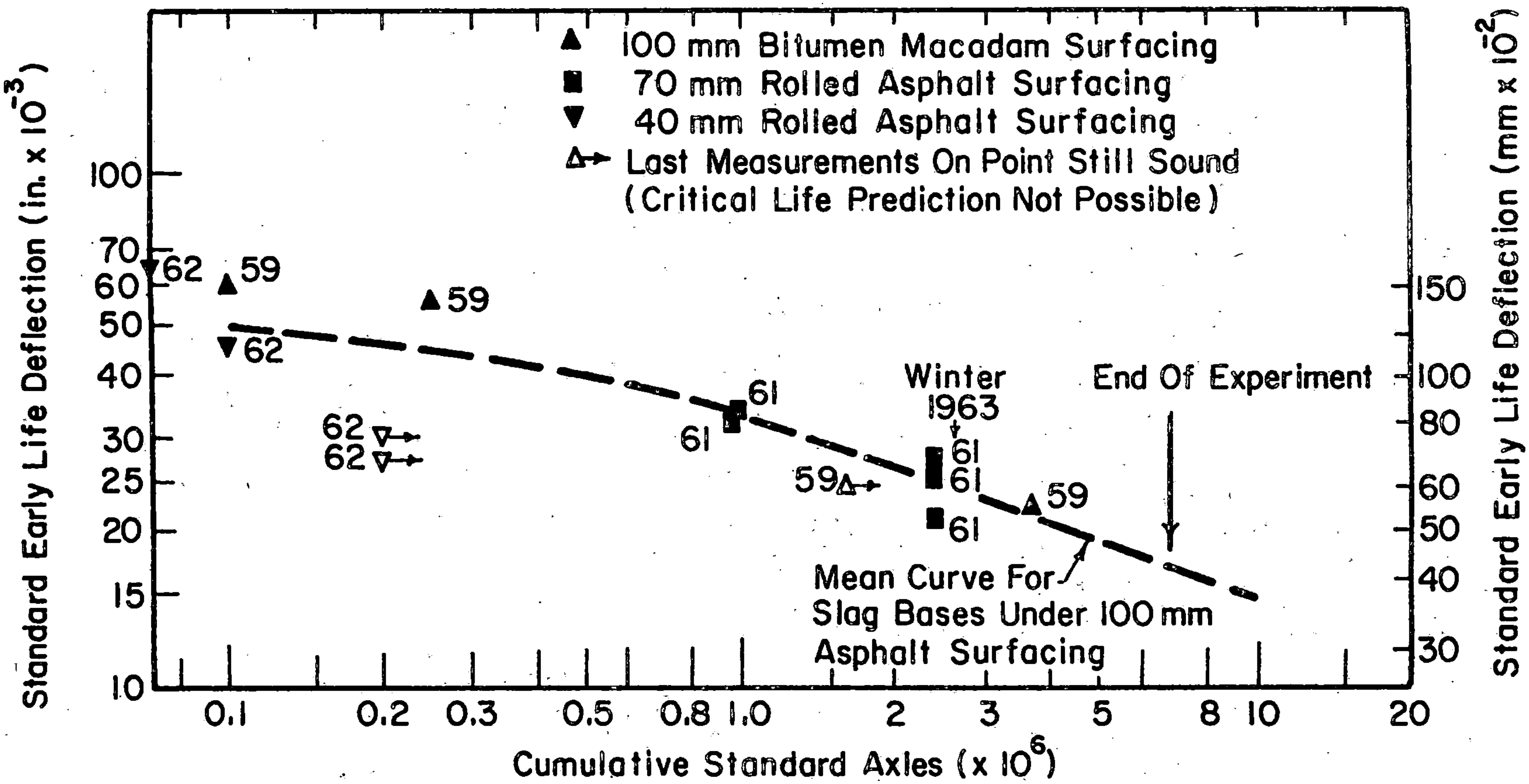


Figure 7.1. Relations Between Deflection and Ctitical Life For Wet-Mix Slag Baees at Alconbury Hill (From Ref. 7.2).

For the spring normal deflections, the equations developed for the AASHO Road Test data were

$$\log w_{2.5} = 9.40 + 1.32 \log L_1 - 3.25 \log d_{sn} \qquad (7.3)$$

$$\log w_{1.5} = 10.18 + 1.36 \log L_1 - 3.64\, d_{sn} \qquad (7.4)$$

where:

d_{sn} = spring normal deflection, in 0.001 in. (Spring normal deflection is deflection value obtained during spring of year by taking difference between probe elevation before loading and when load is opposite the probe.

It was also demonstrated at the Brampton Test Road in Canada that the subsequent performance of a pavement can be predicted from the knowledge of its initial peak Benkelman Beam rebound (deflection) value (7.4). Deflection measurements are also used in several states for the overlay design of flexible pavement (7.5).

Similarly, the performance of rigid (concrete) pavements at the AASHO Road Test could be predicted with surface deflection (corner and edge positions) measurements.

b. Vertical Stress on Subgrade

During the AASHO Road Test it was found that there existed a critical vertical stress in the subgrade between 9 and 11 psi (62 and 76 kN/m^2), beyond which rutting in the flexible pavements rapidly increased and below which it remained essentially constant (7.6). In other words rutting occurred in the AASHO Road Test subgrade soil when vertical subgrade stresses were in excess of about 9 to 11 psi (62 to 76 kN/m^2). The existence of critical stress levels for subgrade soils was also found in the laboratory by Ahmed and Larew (7.7), and Seed and his colleagues (7.8) who conducted repeated loading tests on subgrade soil specimens at different stress levels. It was found that for the soils tested, the resilient strain remained relatively constant with increasing number of load application but the total axial strain was a function of both the stress level and the number of load applications. Similar results were obtained by the investigators at the British Railways and they refer to the critical stress as the threshold stress (7.9)

c. Shearing Strains on the Subgrade

In a study conducted at the U. S. Army Engineers Waterways Experiment Station, good correlations were obtained between performance of flexible pavement field test sections and the computed maximum shearing strains at the surface of the subgrade soil (7.10). Figure 7.2 shows the relationship between the computed maximum shearing strains and "performance" of several test sections.

d. Vertical Subgrade Strain and Subgrade Deflection

Strong correlations have been developed between vertical subgrade strain in highway pavements and the performance of the pavement. Dorman and Metcalf of Shell used the results of the AASHO Road Test to develop a relationship shown in Figure 7.3, between allowable compressive strain in the subgrade and the number of load applications (7.11). Similar relationships have been obtained by the California Division of Highways between vertical compressive strain and the Traffic Index (an index used to represent total traffic) (7.12). The Asphalt Institute has adopted the use of vertical subgrade strain as one of the criteria in the design of full depth asphalt pavements for air carrier airports (7.13). A limiting value of vertical subgrade strain of 2.4×10^{-4} in./in. has been suggested for pavement design purposes by Deen and others (7.14).

Recently efforts have been made to correlate subgrade deflection to performance. Jung and Phang (7.15) report that theoretical examination of successful Ontario pavement designs revealed that the most promising design criterion for flexible pavements was the vertical deflection at top of the subgrade.

e. Stress Level in Granular Materials

As pointed out in Chapter 2 of this report, the repeated load response of granular materials is dependent on a number of factors. The resilient or elastic response of granular materials appears to remain fairly constant with increased load applications, N, but is a function of the stress state. Permanent strain accumulation in general increases with N but is also greatly dependent on the stress state. Granular materials appear to display a "critical" level of stresses beyond which the rate of plastic or permanent strain development increases with increasing N.

Current Status of Transfer Functions for Railway Support System Design

A review of railway literature indicates that "transfer functions" per se, have never been developed; rather various design criteria have been established. In other words limiting structural behavior criteria based on given levels of performance have not been developed.

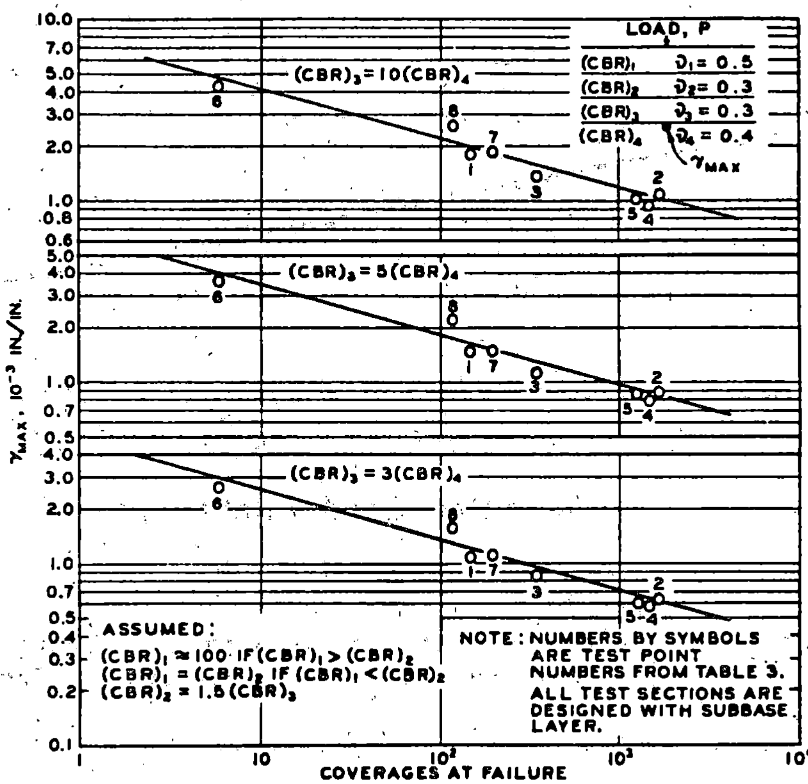


Figure 7.2. Relations Between Maximum Shearing Strains and Performance for Test Sections at Various CBR Ratios (From Ref. 7.10).

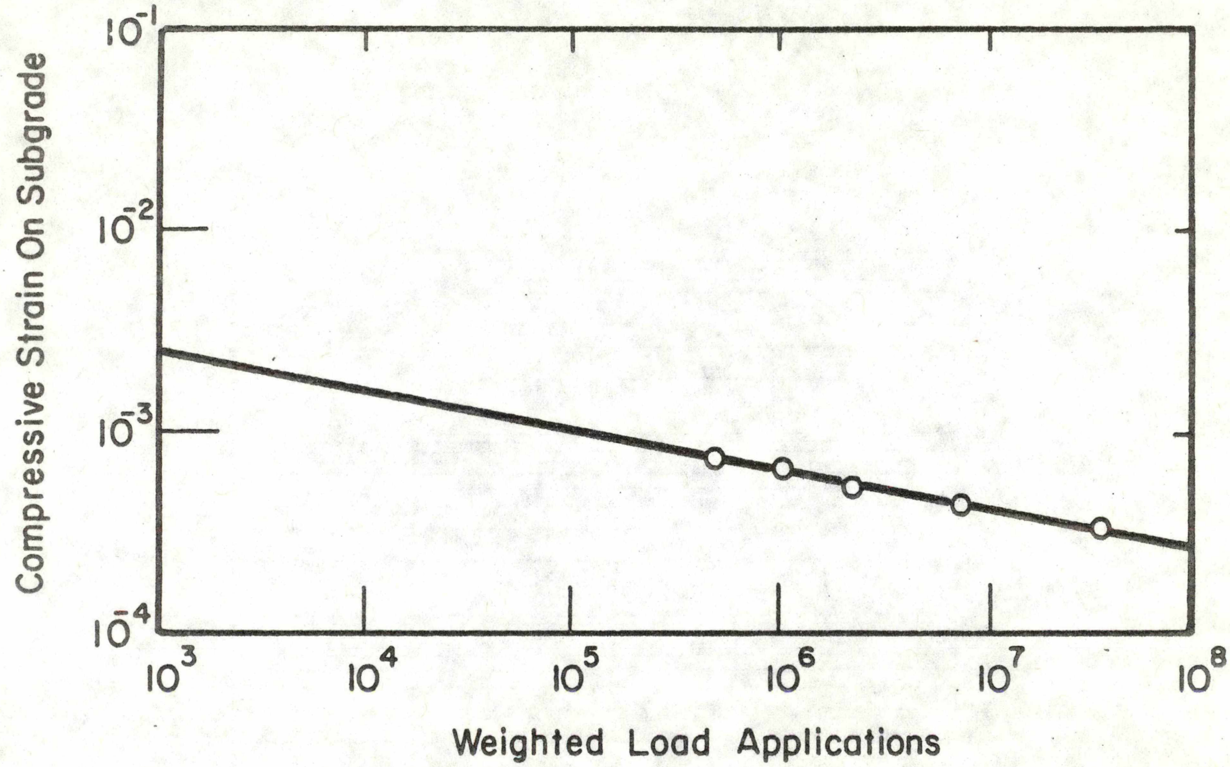


Figure 7.3. Relation of Compressive Strain to Number of Load Applications, 18,000-lb. Axle Load (From Ref. 7.11).

CURRENT TRACK SUPPORT SYSTEM DESIGN CRITERIA

Criteria that are presently used or recommended for use in the design of the railway support structure basically fall into five categories:

1. Allowable rail bending stress (at bottom of rail)
2. Allowable rail deflection.
3. Allowable ballast pressure
4. Allowable subgrade or foundation pressure
5. Allowable subgrade or foundation strain

Allowable Rail Bending Stress

A number of recommended design criteria have been suggested in which the magnitude of bending stress at the bottom of the rail is controlled. Table 7.1 summarizes these various recommendations. It is noted that U. S., British and German recommended criteria are included. The criterion recommended by Magee (7.16) is not in complete agreement with the other criteria but is currently recommended by AREA.

In general, all the procedures have as a basis the concept of rail fatigue. By limiting the magnitude of bending stress under loading, it is implied that fatigue cracking will not occur. It is interesting to note that the magnitude of load and traffic volume are not considered. It appears that only the German procedure (7.16) takes into account the class of track and type of traffic.

The general approach for arriving at an allowable calculated bending stress in the rail is summarized in the following equation (7.17):

$$p_r = \frac{p_y - p_t}{(1 + A)\ (1 + B)\ (1 + C)\ (1 + D)} \qquad (7.5)$$

where:

p_r = allowable calculated rail bending stress
p_y = yield strength of rail steel
p_t = temperature stress in rail
A = stress factor to account for lateral bending of rail
B = stress factor to account for track condition

Table 7.1 Criteria For Calculating Allowable Bending Stress in Rail (From Ref. 7.16)

Stress Factor	Hay	Clark (British)	Magee[1]	German[2] (Class I Track)
Locomotive	5%	5%	0%	
Lateral bending	15%	15%	20%	44%
Temperature	7000 psi (Jointed)	7000 psi (Jointed)	5000 psi (B-J) 10,000 psi (M-J) 20,000 psi (CWR)	40%
Rail Wear	10%	10%	15%	—
Unbalanced Superelevation	15-20%	25%	15%	—
Track Condition	25%	25%	25% (M) 35% (B)	—
Rail Yield Strength, psi	60,000	60,000	70,000	48,000 to 74,000
Wheel Flat Stresses	—	—	—	46%

1: B-J = Branch-Jointed Track

M-J = Main-Jointed Track

CWR = Continuous Welded Rail

2: German rail design practice limits flexural stress based on the class of track and type of traffic.

C = stress factor to account for rail wear and corrosion

D = stress factor to account for unbalanced elevation

Allowable Rail Deflection

AREA (7.17) recommends that the rail deflection under load be restricted to less than 0.25 in. (0.63 cm).

Allowable Ballast Pressure

Another design criteria that has been recommended by AREA (7.17) is that the allowable ballast pressure under load be a maximum of 65 psi (448 kN/m^2).

Allowable Subgrade Stress

There are a number of tentative allowable subgrade stress design criteria. AREA (7.17) has recommended a value less than 20 psi (138 kN/m^2). British Railway (7.9) uses the concept of "threshold stress" which is defined as that magnitude of stress above which permanent plastic deformation accumulates rapidly with additional load applications and below which the rate of permanent deformation in the foundation or subgrade is very small. This "threshold stress" level is determined using a repeated triaxial testing procedure. It appears that most subgrade soils have a threshold stress less than 20 psi (138 kN/m^2).

Ireland (7.18) recommended a "design" approach for determining ballast thickness which is based on Boussinesq stress distribution, static loading conditions, and the strength of the subgrade soil. Basically, the procedure requires enough ballast such that "undue creep or bearing capacity failure" is precluded. Figure 7.4 represents the recommended design approach. It should be pointed out that a factor of safety between 1.5 and 2.0 is recommended. If one works through the procedure, it is found that the allowable stress on the subgrade is about 1.3 to 1.7 times the unconfined compressive strength depending on the factor of safety used.

Early work by Talbot (7.19) simply recommended that the ballast be thick enough so that the pressure transmitted to the subgrade is uniform. No mention however is made of the "allowable magnitude of subgrade stress."

Allowable Subgrade Strain

Heath et al. (7.20) recommend that the subgrade stress be limited to a value so that 10 percent strain occurs in the laboratory tested samples

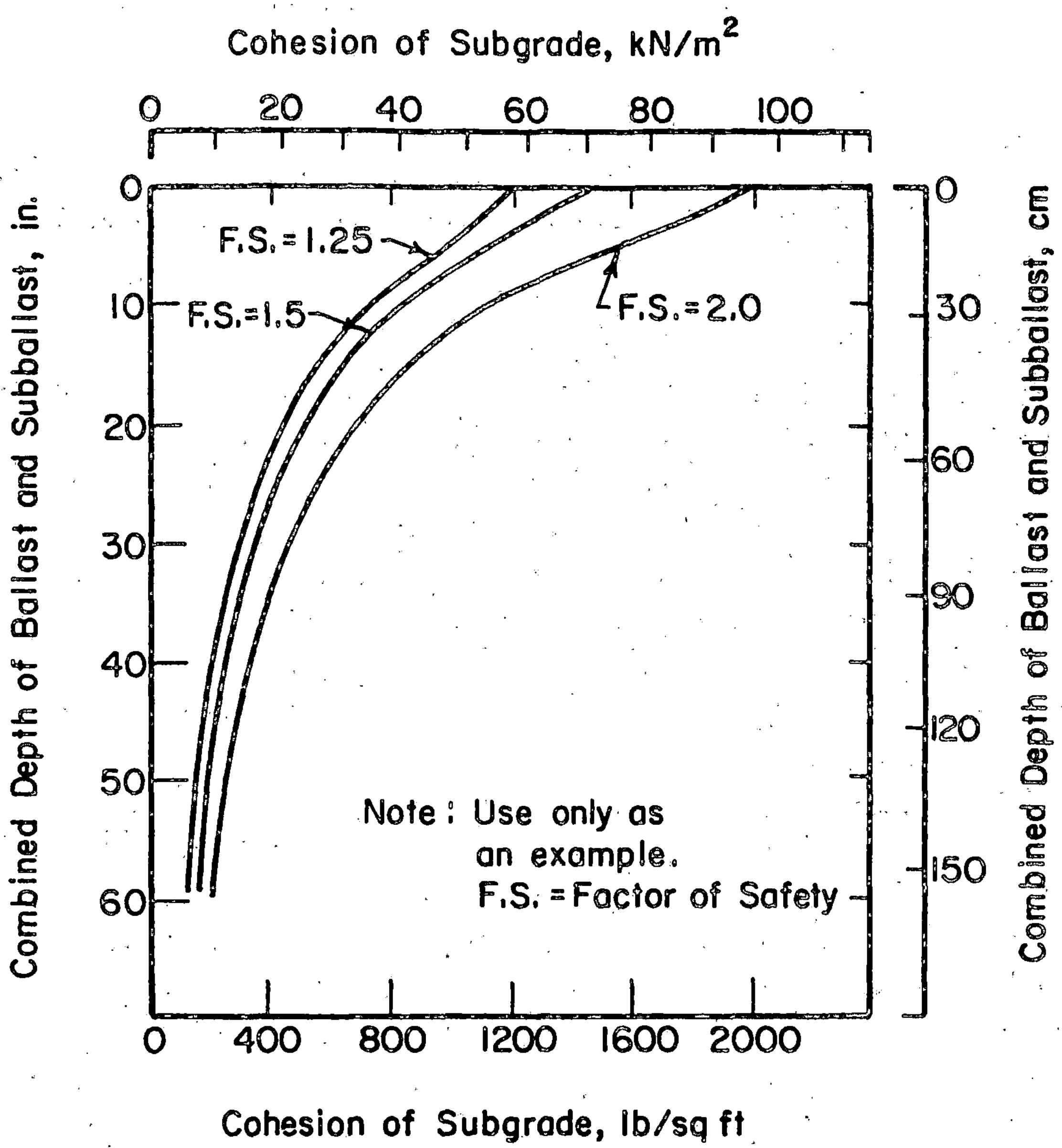


Figure 7.4. Approximate Design Curves for Ballast Thickness (From Ref. 7.18).

of the subgrade soil after more than one million load applications of that stress.

TECHNIQUES FOR ESTIMATING PERMANENT DEFORMATION IN BALLAST AND ROADBED SECTIONS

In this section, the techniques for estimating permanent deformation (generally termed rutting) in ballast and roadbed will be discussed. The permanent deformation in the track system takes place as a result of progressive accumulation of plastic strain in the ballast and the roadbed due to the applied traffic loading.

Permanent Deformation in the Roadbed

The methods for "restricting" the permanent deformation in the roadbed were discussed previously. The permanent deformation of the roadbed can be restricted to tolerable levels if the applied stress level in the roadbed is below some tolerable stress level. However, the estimation of the amount of permanent deformation contributed by the roadbed to the total in the track support system requires establishement of relationships between permanent strain, applied stress and number of stress applications based on laboratory repeated load triaxial compression tests (7.21).

For fine grained soils, the relationship between ε_p and N is of the form (7.21):

$$\varepsilon_p = AN^b \tag{7.6}$$

and at a particular number of stress applications

$$\Delta\sigma = \frac{\varepsilon_p}{1 + m\varepsilon_p} \tag{7.7}$$

where:

ε_p = permanent strain

N = number of stress repetitions

$\Delta\sigma$ = applied stress

A,b,m = experimentally determined coefficients

Monismith, et al. (7.21) have suggested the use of "strain-hardening" and "time-hardening" procedures to evaluate the effects of cumulative loading.

They report however that these procedures give results which agree only qualitatively with experimental data.

Permanent Deformation in Granular Materials

Barksdale (7.22) has presented a methodology for estimating rut depth using plastic material properties evaluated from repeated load triaxial tests together with either linear or nonlinear elastic layered theory. The layered structure is divided into several fictitious sublayers and the major principal stress, σ_1, and average confining pressure, σ_3, are calculated at the center of each sublayer using a structural behaviour theory. The plastic strain at those points is interpolated from repeated load triaxial test results for the desired number of load applications which for unstabilized granular material is of the form (7.23).

$$\varepsilon_p = \frac{(\sigma_1 - \sigma_3)/(K\sigma_3^{\,n})}{1 - \dfrac{(\sigma_1 - \sigma_3)R_f\,(1 - \sin\phi)}{2(C\cos\phi + \sigma_3\sin\phi)}} \qquad (7.8)$$

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

The total rut depth beneath the applied load in the layered structure can then be obtained by summing all of the products of the average plastic strains occurring at the center of each sublayer and the corresponding sublayer thickness, viz.,

$$\delta^p_{total} = \sum_{i=1}^{n} (\varepsilon_i^p\, h_i) \qquad (7.9)$$

where

δ^p_{total} = total rut depth beneath the wheel load

ε^p_i = average plastic strain in the i^{th} subgrade

h_i = thickness of the i^{th} sublayer

n = total number of sublayers

Barksdale (7.22) has also suggested a method for simple evaluation and comparison of the rutting tendencies of unstabilized granular material by using the Rut Index. He defines the Rut Index as the sum of the average plastic strains occurring in the top and bottom half of the base multiplied by an arbitrarily chosen factor of 10,000 so as to give a whole number.

The Office of Research and Experiments(7.24) have reported the development of a relationship by which the permanent deformation behavior of ballast can be predicted:

$$e_N = 0.082\ (100n - 38.2)\ (\sigma_1 - \sigma_3)^2 (1 + 0.2 \log N) \tag{7.10}$$

where

e_N = permanent strain after N loading cycles

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

n = initial porosity

N = number of repeated loading cycles

CHAPTER 8

SUMMARY AND CONCLUSIONS

The primary objective of this report has been to summarize the current technology relative to the following:

1. Procedures and techniques for evaluating ballast and subgrade material properties (shear strength, resilience, susceptibility to plastic deformation, capacity for lateral and longitudinal restraint, etc.)
2. Factors which significantly influence material properties (water content, density, durability, loading conditions, gradations, etc.)
3. Relations between material properties and track system response and performance. Identification of the cause and development of failures and other useful performance criteria, performance indices, or material parameters required to elude failure.
4. Review and application of "structural models" such as those of Meacham, Lundgren, and others relative to predicting the behavior of the track support system (subgrade-ballast-tie-rail system).
5. Identify "transfer functions" which relate track system response stress or strain levels in materials and subgrades, etc. to performance.

Based on the information and literature reviewed and the discussion presented in this report, the following conclusions appear warranted:

1. Ballast and subgrade soil materials receive inadequate consideration in the analysis and design of conventional track support structures.
2. Strength properties of ballast and roadbed soil materials are rarely determined.
3. Numerous quality tests are normally conducted on ballast materials. In general, few correlations have been developed between quality test results and field performance of ballast materials.
4. Stress-strain characteristics of ballast and subgrade soil materials should be evaluated using testing procedures which closely simulate actual in-service loading conditions. Currently, it appears that the repeated load triaxial test can be used to adequately evaluate

the "strength" characteristics of ballast and subgrade materials. Typical characterization includes determination of resilient and permanent deformation behavior under repeated loading.

5. Numerous factors significantly affect the repeated load response of ballast and subgrade soil materials. Therefore, it is necessary to establish the magnitude and range of such factors and appropriately consider these in the process of evaluating material properties.
6. "Structural models" presently available for structural analysis of the track support system do not adequately consider the "stress dependent" nature of ballast and subgrade soil materials.
7. Transfer functions for relating track system structural response to track system performance presently are not available. Various limiting structural behavior criteria have been recommended for use in design of the track support system.
8. More realistic and valid structural behavior theories are available in the highway and airfield pavements field. With appropriate modifications these theories could be used for more realistic evaluation of the structural response of the railway support system.
9. Appropriate consideration must be given to climatic factors (such as temperature and moisture) in the analysis and design of the railway support system.

2.12 Report of Committee 1, Proceedings, American Railway Engineering Association, Volume 58, 1957, pp. 748-773.

2.13 Butler, A. B., "An Analysis of Bending Stress and Deflections in Railroad Rails," M.Sc. Thesis, Civil Engineering Department, University of Illinois, Urbana, 1969.

2.14 Prause, R. H., et al., "Assessment of Design Tools and Criteria for Urban Rail Track Structures - Volume I, At-Grade Tie-Ballast Track," Urban Mass Transportation Administration, Department of Transportation, Report No. UMTA-MA-06-0025-74-3, April 1974.

2.15 FRA Track Safety Standards, Federal Railroad Administration, 1972.

2.16 Dempsey, B. J., Unpublished Data.

2.17 Lind, J. F., and Luttrell, N. W., Rail-Vehicle Interaction Study, Southern Pacific Transportation Company, April, 1971.

2.18 Report of Committee 1, Roadway and Ballast, Bulletin 651, A.R.E.A., January-February, 1975.

2.19 Weber, John W., "Development of the Prestressed Concrete Tie in the U.S.," paper presented at the Symposium on Railroad Track Mechanics, Princeton University, April, 1975.

2.20 "Adhesive in Rail-Joint Assemblies," Railway Gazette, Vol. 109, December, 1958.

2.21 Spang, J., "Protective Blankets of Track Formations," Archiv F. Esenbahntechnik, No. 20, 1965.

REFERENCES

Chapter 2

2.1 Manual for Railway Engineering, American Railway Engineering Association, 1973.

2.2 "Construction and Maintenance Operations," Proceedings, American Railway Engineering Association, Volume 75, Bulletin 645, November-December, 1973.

2.3 Minerals Yearbook, 1972, Bureau of Mines, U. S. Department of the Interior, Vol. I, Published 1974.

2.4 Peckover, F.L., "Railway Ballast Material," Office of Chief Engineer, Canadian National Railways, Montreal, Quebec, June 29, 1973.

2.5 Hay, W. W., Railroad Engineering, Vol. 1, John Wiley and Sons, Inc., New York, 1953.

2.6 A.R.E.A. Bulletin 651, January-February, 1975, pp.282-297.

2.7 Meacham, H.C., et al., "Studies for Rail Vehicle Track Structures," Office of High Speed Ground Transportation, Federal Railroad Administration, Report No. FRA-RT-71-45, April 30, 1970.

2.8 Talbot, A. N., "Stresses in Railroad Track," Reports of the Special Committee to Report on Stresses in Railroad Track"
First Progress Report, Proceedings, AREA, Vol. 19, 1918, pp. 873-1062.
Second Progress Report, Proceedings, AREA, Vol. 21, 1920, pp. 645-814.
Third Progress Report, Proceedings, AREA, Vol. 24, 1923, pp. 297-453.
Fourth Progress Report, Proceedings, AREA, Vol. 26, 1925, pp. 1081-1245.
Fifth Progress Report, Proceedings, AREA, Vol. 31, 1930, pp. 69-336.
Sixth Progress Report, Proceedings, AREA, Vol. 35, 1934, pp. 1148-1155.

2.9 Peterson, L. A., et al., "Measurement and Analysis of Wheel-Rail Forces," American Society of Mechanical Engineering Publication 71-WA/RT-4, August 2, 1971.

2.10 Engineering and Design, Pavement Design for Frost Conditions, Corps of Engineers, EM-1110-1-306, May 15, 1962.

2.11 Dempsey, B. J. and Thompson, M.R., "A Heat-Transfer Model for Evaluating Frost Action and Temperature Related Effects in Multilayered Pavement Systems," Highway Research Record, No. 342, Transportation

Chapter 3

3.1 Peckover, F. L., "Railway Ballast Material," Office of Chief Engineer, Canadian National Railways, Montreal, Quebec, June, 1973.

3.2 West, R. T., et al., "Tests for Evaluating Degradation Characteristics of Base Course Aggregates," National Cooperative Highway Research Program Report 98, 1970.

3.3 Bloem, D. L., "Soundness and Deleterious Substances," ASTM STP No. 169-A, 1966.

3.4 Dalton, C. J., "Field Durability Tests on Ballast Samples as a Guide to the Significance of the Specification Requirements," Technical Research Centre, Canadian National Railways, Montreal, Quebec, January, 1973.

3.5 Raymond, G. P., et al., "A Study of Stresses and Deformations under Dynamic and Static Load Systems in Track Structure and Support," Canadian Institute of Guided Ground Transport, January, 1975.

3.6 AASHTO Materials, 11th ed., American Association of State Highway and Transportation Officials, Washington, D.C., 1974.

3.7 Manual for Railway Engineering, American Railway Engineering Association, 1973.

3.8 Scalzo, P. A., "Laboratory Testing of Railway Ballast," Queen's University, Kingston, Ontario, April, 1972.

3.9 Goldbeck, A. T., "Mineral Aggregates for Railroad Ballast," ASTM STP No. 83, 1948.

3.10 British Standard 812, Methods for Sampling and Testing of Mineral Aggregates, Sands, and Filters, British Standards Institution, June 1967.

3.11 Rad, P. F., "A Simple Technique for Determining Strength of Aggregates," ASTM Symposium, Atlanta, Georgia, 1974.

3.12 Annual Book of ASTM Standards, American Society for Testing and Materials, Part 19, Philadelphia, 1974.

3.13 Huang, E. Y., "A Test for Evaluating the Geometric Characteristics of Coarse Aggregate Particles," ASTM Proceedings, Vol. 62, 1962.

3.14 Huang, E. Y., "An Improved Particle Index Test for the Evaluationg of Geometric Characteristics of Aggregates," Michigan Highway Research Project No. 86546, Houghton, Michigan, July, 1965.

3.15 Huang, E. Y., Unpublished Report, 1974.
Research Board, 1970.

3.16 Ishai, I., and E. Tons, "New Concept and Test Method for a Unified Characterization of the Geometric Irregularity of Aggregate Particles," private communication.

3.17 Bérard, J., "Petrographic Study of Ballasts," Ecole Polytechnique, Montreal, Quebec, September, 1973.

3.18 Horn, H. M., and D. U. Deere, "Frictional Characteristics of Minerals," Geotechnique, Vol. XII, No. 4, December, 1962.

3.19 Marachi, et al., "Strength and Deformation Characteristics of Rockfill Materials," Report No. TE-69-5 to State of California Department of Water Resources, Berkeley, California, September, 1969.

3.20 Gray, J. E., "Characteristics of Graded Base Course Aggregates Determined by Triaxial Tests," National Crushed Stone Association, Engineering Bulletin No. 12, July, 1962.

3.21 Klugar, K., "A Contribution Towards Ballast Mechanics," Eisenbahntechnische Rundshau, Vol. 12, No. 8, (translated), August, 1963.

3.22 Pike, D. C., "Shear-box Tests on Graded Aggregates," Transport and Road Research Laboratory, Crowthorne, Berkshire, 1973.

3.23 Peckover, F. L., private communication, 1974.

3.24 Yoder, E. J., Principles of Pavement Design, John Wiley & Sons, New York, N.Y., 1959.

3.25 Huang, E. Y., and D. G. Lohmeier, "Comparison of Soil-Aggregate Mixture Strength by Two Methods," Highway Research Record No. 52, 1964.

3.26 Barksdale, R. D., "Repeated Load Test Evaluation of Base Course Materials," Georgia Institute of Technology, Atlanta, Georgia, May, 1972.

3.27 Hicks, R. G., and C. L. Monismith, "Factors Influencing the Resilient Response of Granular Materials," Highway Research Record No. 345, 1971.

3.28 Kalcheff, I. V., "Characteristics of Graded Aggregates as Related to their Behavior under Varying Loads and Environments," Conference on Utilization of Graded Aggregate Base Materials in Flexible Pavements, Oak Brook, Illinois, March, 1974.

3.29 Klugar, K., "Ballast Deformation," Eisenbahntechnische Rundshau, Vol. 15, No. 11, (translated), 1965.

3.30 Brown, S. F., and A. F. L. Hyde, Unpublished Report.

3.31 Allen, J. J., "The Effects of Nonconstant Lateral Pressures on the Resilient Properties of Granular Materials," Ph.D. Thesis, University of Illinois, Urbana, Illinois, 1973.

3.32 Allen, J. J., and M. R. Thompson, "Significance of Variably Confined Triaxial Testing," Proceedings, ASCE, TE 4, November, 1974.

3.33 Seed, H. B., et al., "Prediction of Flexible Pavement Deflections from Laboratory Repeated-Load Tests," National Cooperative Highway Research Program Report 35, 1967.

3.34 Wong, R. C. T., "One Dimensional Repeated Loading Test of Railway Ballast," M.S. Thesis, Queen's University, Kingston, Ontario, 1974.

3.35 Tice, J. A., "Soil Resilience Test for Virginia Flexible Pavement Design Method," Virginia Highway Research Council, Charlottesville, Virginia, August, 1969.

3.36 Haynes, J. H., and E. J. Yoder, "Effects of Repeated Loading on Gravel and Crushed Stone Base Course Materials Used in the AASHO Road Test, " Highway Research Record 39, 1963.

3.37 Holubec, I., "Cyclic Creep of Granular Materials," D. H. O. Report No. RR147, University of Waterloo, Waterloo, Ontario, June, 1969.

3.38 Snyder, J. R., "Repeated Triaxial Testing Machine for Stone with Experimental Evaluations," M. S. Thesis, West Virginia University, Morgantown, West Virginia, 1964.

3.39 Coffman, B. S., et al., "A Comparison of Calculated and Measured Deflections for the AASHO Test Road," Proceedings, Association of Asphalt Paving Technologists, Vol. 33, Dallas, Texas, 1964.

3.40 "The Kansas Test Track - A Progress Report," Rail Technology Division,Federal Railroad Administration, Report No. FRA-RT-73-08, October, 1971.

3.41 "Stresses in the Rails, the Ballast, and in the Formation Resulting From Traffic Loads," Question D71, Office of Research and Experiments of the International Union of Railways, Utrecht, October, 1970.

3.42 "Study of the Change in the Track Level as a Function of the Traffic and of the Track Components," Question D117, Office of Research and Experiments of the International Union of Railways, Utrecht, April, 1973.

3.43 Woolf, D. O., "Toughness, Hardness, Abrasion, Strength, and Elastic Properties," ASTM STP No. 169-A, 1966.

3.44 Powers, T. C., "Geometric Properties of Particles and Aggregates," Journal of the PCA Research and Development Laboratories, January, 1964.

3.45 Townsend, D. L., and J. W. Madill, "The Effect of Crushed Particles on the Strength of Base Course Materials," Queen's University, Kingston, Ontario, August, 1964.

3.46 Vallerga, B. A., et al., "Effect of Shape, Size and Surface Roughness of Aggregate Particles on the Strength of Granular Materials," ASTM STP No. 212, 1957.

3.47 Ferguson, E. G., and J. M. Hoover, "Effect of Portland Cement Treatment of Crushed Stone Base Materials as Observed from Triaxial Shear Tests," Highway Research Record, No. 255, 1968.

3.48 Chung, K. M. P., "Some Triaxial Tests on Railway Ballast," Queen's University, Kingston, Ontario, 1974.

3.49 Bishop, A. W., and G. E. Green, "The Influence of End Restraint on the Compression Strength of a Cohesionless Soil," Geotechnique, Vol. XV, No. 3, September, 1965.

3.50 Holtz, W. G., and H. J. Gibbs, "Triaxial Shear Tests on Pervious Gravelly Soils," Proceedings, ASCE, January, 1956.

3.51 Leslie, D. D., "Large Scale Triaxial Tests on Gravelly Soils," Proceedings, 2nd Pan American Conference on Soil Mechanics and Foundation Engineering, 1963.

3.52 Smith, W. S., and K. Nair, "Development of Procedures for Characterization of Untreated Granular Base Course and Asphalt-Treated Base Course Materials," Federal Highway Administration Report FHWA-RD-74-61, October, 1973.

3.53 Thompson, O. O., "Evaluation of Flexible Pavement Behavior with Emphasis on the Behavior of Granular Layers," Ph.D. Thesis, University of Illinois, Urbana, Illinois, 1969.

3.54 Duncan, J. M., and C. Y. Chang, "Nonlinear Analysis of Stress and Strain in Soils," Proceedings, ASCE, Vol. 96, SM5, September, 1970.

3.55 American Railway Signaling Principles, Chapter VII, Non-Loaded Direct Current Track Circuits, Signal Section, A.A.R., 1948.

3.56 "Advanced Report of Committee 1 - Roadway and Ballast Report on Assignment 10 (a), Proceedings, A.R.E.A., Vol. 64, 1963.

Additional References

"Annotated Bibliography on Soil - Aggregate Mixtures," Illinois Cooperative Highway Research Program, Parts I and II, Urbana, Illinois.

"Annotated Bibliography on Track Ballast," Canadian National Railways, Special Series No. 43, Montreal, Quebec.

<u>Chapter 4</u>

4.1 Robnett, Q. L., and M. R. Thompson, Unpublished data, Department of Civil Engineering, University of Illinois, Urbana, Illinois, 1975.

4.2 Seed, H. B., and C. K. Chan, "Effect of Duration of Stress Application on Soil Deformation Under Repeated Loading," <u>Proceedings</u>, Fifth International Conference on Soil Mechanics and Foundation Engineering, Vol. I, 1964, pp. 341-345.

4.3 Seed, H. B., and C. K. Chan, "Effect of Stress History and Frequency of Stress Application on Deformation of Clay Subgrades Under Repeated Loading," <u>Proceedings</u>, Highway Research Board, Vol. 37, 1958, pp. 555-575.

4.4 Seed, H. B., and C. K. Chan, "Structure and Strength Characteristics of Compacted Clays," <u>Transactions</u>, American Society of Civil Engineers, Vol. 126, Part I, 1961, pp. 1344-1384.

4.5 Seed, H. B., and C. K. Chan, "Thixotropic Characteristics of Compacted Clays," <u>Transactions</u>, American Society of Civil Engineers, Vol. 124, 1959, pp. 894-925.

4.6 Seed, H. B., C. K. Chang, and C. E. Lee, "Resilience Characteristics of Subgrade Soils and their Relation to Fatigue Failures in Asphalt Pavements," <u>Proceedings</u>, First International Conference on Structural Design of Asphalt Pavements, Ann Arbor, Michigan, 1962, pp. 611-636.

4.7 Seed, H. B., C. K. Chan, and C. L. Monismith, "Effects of Repeated Loading on the Strength and Deformation of Compacted Clay," <u>Proceedings</u>, Highway Research Board, Vol. 34, 1955, pp. 541-558.

4.8 Seed, H. B., and R. L. McNeill, "Soil Deformations in Normal Compression and Repeated Loading Tests," <u>Bulletin 141</u>, Highway Research Board, 1956.

4.9 Seed, H. B., and R. L. McNeill, "Soil Deformation Under Repeated Stress Applications," <u>Special Technical Publication No. 232</u>, American Society for Testing Materials, 1958, pp. 177-197.

4.10 Seed, H. B., R. L. McNeill, and J. DeGuenin, "Clay Strength Increase Caused by Repeated Loading," <u>Transactions</u>, American Society of Civil Engineers, Vol. 125, 1960, pp. 141-165.

4.11 Seed, H.B., F. G. Mitry, C. L. Monismith, and C. K. Chan, "Prediction of Flexible Pavement Deflections from Laboratory Repeated Load Tests," National Cooperative Highway Research Program, <u>Report 35</u>, Highway Research Board, 1967.

4.12 Seed, H. B., and C. L. Monismith, "Strength Evaluation of Pavement Structure Elements," Proceedings, First International Conference on Structural Design of Asphalt Pavements, Ann Arbor, Michigan, 1962, pp. 551-566.

4.13 Papazian, H. S., "The Response of Linear Viscoelastic Materials in the Frequency Domain with Emphasis on Asphalt Concrete," Proceedings, First International Conference on Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, Michigan, 1962.

4.14 Coffman, B.S., D. C. Craft, and J. Tamayo, "A Comparison of Calculated and Measured Deflections for the AASHO Road Test," Proceedings, Association of Asphalt Paving Technologists, Vol. 33, 1964, pp. 54-91.

4.15 Seed, H. B., et al., "The Strength of Compacted Cohesive Soils," Proceedings, American Society of Civil Engineers Research Conference on Shear Strength of Compacted Cohesvie Soils, Boulder, Colorado, 1960.

4.16 Casagrande, A. and W. L. Shannon, "Research on Stress Deformation and Strength Characteristics of Soils and Soft Rocks Under Transient Loading," Soil Mechanics Series No. 31, Harvard University, 1948.

4.17 Olson, R. E., and H. Kane, "Dynamic Shearing Properties of Compacted Clay at High Pressures," Proceedings, Sixth International Conference on Soil Mechanics and Foundation Engineering, Montreal, Quebec, 1965.

4.18 Olson, R. E., and J. F. Parola, "Dynamic Shearing Properties of Compacted Clay," Proceedings, International Symposium on Wave Propagation and Dynamic Properties of Earth Materials, Albuquerque, New Mexico, 1967.

4.19 Dehlen, G. L., "The Effect of Non-Linear Material Response on the Behavior of Pavements Subjected to Traffic Loads," Ph.D. Dissertation, University of California, Berkeley, 1969.

4.20 Monismith, C. L., N. Ogawa, and C. R. Freeme, "Permanent Deformation Characteristics of Subgrade Soils in Repeated Loading," paper presented at annual meeting of Transportation Research Board, Washington, D.C., January, 1975.

4.21 Robnett, Q. L. and M. R. Thompson, "Development of Testing Procedure," Phase I, Interim Report, Resilient Properties of Subgrade Soils, Civil Engineering Studies, Transportation Engineering Series No. 5, Series No. 139, May, 1973.

4.22 Pagen, C. A., and V. K. Khosla, "Effect of Freeze-Thaw on Rheological Characteristics of a Compacted Clay," Transportation Research Record No. 497, Transportation Research Board, 1974.

4.23 Bergan, A. T. and D. G. Fredlund, "Characterization of Freeze-Thaw Effects on Subgrade Soils," Symposium on Frost Action on Roads, Report Volume II, Organization for Economic Development, Oslo, Norway, 1973.

4.24 Bergan, A. T. and C. L. Monismith, "Characterization of Subgrade Soils in Cold Regions for Pavement Design Purposes," Highway Research Record No. 431, Highway Research Board, 1973.

4.25 Robnett, Q. L., and M. R. Thompson, "Effect of Lime Treatment on the Resilient Behavior of Fine-Grained Soils," paper presented at 54th Annual Meeting of the Transportation Research Board, January, 1975.

4.26 PCA Soil Primer, Portland Cement Association, 1962.

4.27 Edwards, J. M. and C. P. Valkering, "Structural Design of Asphalt Pavements for Heavy Aircraft," Shell International Petroleum Company, Limited, London, 1970.

4.28 Seed, H. B. and C. K. Chan, "Undrained Strength After Soaking," Transactions, ASCE, Vol. 126, Part 1, 1961.

4.29 Sauer, E. K. and Monismith, C. L., "Influence of Soil Suction on Behavior of a Glacial Till Subjected to Repeated Loading," Highway Research Record No. 215, Highway Research Board, 1968.

4.30 Sherif, M. A. and C. M. Burrous, "Temperature Effects on Unconfined Shear Strength of Saturated, Cohesive Soil," Special Report 103, Highway Research Board, 1969.

4.31 "Investigation of Description, Classification, and Strength Properties of Frozen Soils," SIPRE Report 8, The Frost Effects Laboratory, Corps of Engineers, U. S. Army, Boston, Massachusetts, 1952.

4.32 Vesic, A. B. and Domaschuk, L., "Theoretical Analysis of Structural Behavior of Road Test Flexible Pavements," NCHRP, Report 10, Highway Research Board, 1964.

4.33 Ahmed, S. B. and H. G. Larew, "A Study of Repeated Load Strength Moduli of Soils," Proceedings, First International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, 1962.

Chapter 5

5.1 Talbot, A. N., "Stress in Railroad Track," Reports of the Special Committee to Report on Stresses in Railroad Track.
First Progress Report, Proceedings, AREA, Vol. 19, 1918, pp. 873-1062.
Second Progress Report, Proceedings, AREA, Vol. 21, 1920, pp. 645-814.

5.2 Clark, C. W., "Track Loading Fundamentals - 1," The Railway Gazette, January 11, 1957.

5.3 Magee, G. M., "Analytical Study of Railroad Track," Association of American Railroads Research and Test Department, August, 1966.

5.4 Meacham, H. C., et al., "Studies for Rail Vehicle Track Structures," Office of High Speed Ground Transportation, Federal Railroad Administration, Report No. FRA-RT-71-45, April 30, 1970.

5.5 Hetenyi, M., Beams on Elastic Foundation, University of Michigan Press, 1946.

5.6 Prause, R. H., et al., "Assessment of Design Tools and Criteria for Urban Rail Track Structures - Volume 1, At-Grade Tie-Ballast Track," Office of Research and Development, Urban Mass Transportation Administration, Department of Transportation, Report No. UMTA-MA-06-0025-74-3, April, 1974.

5.7 Barden, Laing, "Distribution of Contact Pressure Under Foundations," Geotechnique, Vol. XIII, No. 3, September, 1962.

5.8 Harrison, H. B., "General Computer Analysis of Beams on Elastic Foundations," Proceedings, The Institution of Civil Engineers, London, Vol. 55, Part 2, September, 1973.

5.9 Rostler, F. S., et al., "Study of Methods of Stabilizing Conventional Ballast Using Polymers," Materials Research and Development, Inc. Oakland, California, December, 1966.

5.10 Newmark, N.M., "Influence Charts for Computation of Stresses in Elastic Foundations," Bulletin Series No. 338, Engineering Experimental Station, University of Illinois, June 1964.

5.11 Ireland, H. O., "Railroad Subgrade Stresses," Proceedings, American Railway Engineering Association, Bulletin 641, January - February, 1973.

5.12 Lundgren, J. R., et al., "A Simulation Model of Ballast Support and the Modulus of Track Elasticity," Civil Engineering Studies, University of Illinois, Urbana, Transportation Series No. 4, September, 1970.

5.13 Annual Report - Part 1: Research Progress Report 1973-74, Canadian Institute of Guided Ground Transportation, Queen's University at Kingston, Ontario, Report No. 73-11, June, 1974.

5.14 "Construction and Maintenance Operations," Proceedings, American Railway Engineering Association, Volume 75, Bulleting 645, November-December, 1973.

5.15 Sinha, Hari, "Ballast in Railway Track," Indian Railway Technical Bulletin, February, 1967.

5.16 Heath, D. L., et al., "Design of Conventional Rail Track Foundations," Proceedings, The Institution of Civil Engineers, London, Vol. 52, February, 1972.

5.17 Heath, D. L., et al., "Discussion, Design of Conventional Rail Track Foundations," Proceedings, The Institution of Civil Engineers, London, Vol. 53, Part 2, December, 1972.

5.18 Heath, D. L. and M. Cottram, "Stresses Beneath a Railway Track," The Railway Gazette, Vol. 122, December 16, 1966.

5.19 Walters, J. M. and M. J. Shenton, "Track Foundation Design," The Railway Gazette, Vol. 124, October 4, 1968.

5.20 AREA Manual for Railway Engineers, American Railway Engineers Association, 1973.

5.21 "The Kansas Test Track - A Progress Report," Rail Technology Division, Federal Railroad Administration, Report No. FRA-RT-72-08, October, 1971.

Chapter 6

6.1 Hardy, R. M., "Stress Distribution in Track Structure," Proceedings, American Railway Engineering Association, Vol. 69, Bulleting 614, June-July, 1968.

6.2 Kamath, K. N., and A. K. Mukerjee, "Investigation on Intensity of Pressure on the Formation - Static Tests at RDSO," Indian Railway Technical Bulletin, November, 1972.

6.3 "Study of the Change in Track Level as a Function of the Traffic and of the Track Components," Office of Research and Experiments of the International Union of Railways, Question D117, Report No. 2, April, 1973.

6.4 Salem, M. T., "Vertical Pressure Distribution in the Ballast Section and on the Subgrade Beneath Statically Loaded Ties," Ph.D. Thesis, Civil Engineering Department, University of Illinois, Urbana, 1966.

6.5 Birman, F., "The Elastic Properties of Railway Ballast," Bundesverband Naturatein Industrie, Issue 42, Vol. 11 (1965), No. 4.

6.6 Eisenman, J. and H. Gnad, "Effect of Ballast Conditions on Track Stability, "The Railway Gazette, May 1, 1970.

6.7 Klugar, K., "A Contribution Towards Ballast Mechanics," Eisenbahntechnische Rundschau, Vol. 12, No. 8.

6.8 Rostler, F. S., et al., "Study of Methods of Stabilizing Conventional Ballast Using Polymers," Materials Research and Development, Inc. Oakland, California, December, 1966.

6.9 Magee, G. M., "Stabilized Ballast Investigations," The Research Center, Association of American Railroads, August, 1969.

6.10 "Asphalt Treatment Reduces Roadbed Maintenance," Information Series No. 126, The Asphalt Institute, October, 1963.

6.11 Peck, R. B., "Progress Report of the Joint Investigation of Methods of Roadbed Stabilization," Bulletin, University of Illinois, Vol. 43, No. 66, July 6, 1946.

6.12 Darter, M. I., "Application of Statistical Methods to the Design of Pavement Systems," A paper presented at the 54th Annual Meeting of the Transportation Research Board, January, 1975.

6.13 Darter, M. I., et al., "Statistical Variation of Flexible Pavement Properties and Their Consideration in Design," Proceedings, The Association of Asphalt Paving Technologists, Vol. 42, 1973.

6.14 Levey, J. R., and Barenberg, E. J., "A Procedure for Evaluating Pavements with Non-Uniform Paving Materials," Highway Research Record, No. 337, Transportation Research Board, 1970.

6.15 Barenberg, E. J., et al., "Mathematical Analysis of Pavements with Non-Uniform Materials," A paper prepared for presentation at the 1975 Transportation Research Board Symposium-Role of Probability and Statistics in Structural Analysis of Pavements, January, 1975.

6.16 Butler, A. B., "An Analysis of Bending Stress and Deflections in Railroad Rails," M.Sc. Thesis, Civil Engineering Department, University of Illinois, Urbana, 1969.

6.17 Meacham, H. C., et al., "Studies for Rail Vehicle Track Structures," Office of High Speed Ground Transportation, Federal Railroad Administration, Report No. FRA-RT-71-45, April 30, 1970.

6.18 Lundgren, J. R., et al., "A Simulation Model of Ballast Support and the Modulus of Track Elasticity," Civil Engineering Studies, University of Illinois, Urbana, Transportation Series No. 4, September, 1970.

Chapter 7

7.1 Boyer, R. E., and M. E. Harr, "Predicting Pavement Performance," Journal, Transportation Engineering, American Society of Civil Engineers, Vol. 100, No. TE2, May, 1974.

7.2 Lister, N.W., "Deflection Criteria for Flexible Pavements," Transport and Road Research Laboratory, Crowthorne, Berkshire, England, TRRL Report LR 375, 1972.

7.3 "The AASHO Road Test, Part 5, Pavement Research," Special Report 61E, Transportation Research Board, 1962.

7.4 Phang, W. A., "Flexible Pavement Design in Ontario," A paper presented at the Annual Meeting of the Transportation Research Board, Washington, D.C., January, 1974.

7.5 Van Til, C. J., et al., "Evaluation of AASHO Interim Guides for Design of Pavement Structures," National Cooperative Highway Research Program Report 128, Transportation Research Board, 1972.

7.6 Vesic, A. S., and L. Domaschuk, "Theoretical Analysis of Structural Behavior of Road Test Flexible Pavements," National Cooperative Highway Research Program Report 10, Transportation Research Board, 1964.

7.7 Ahmed, S. B., and H. G. Larew, "A Study of Repeated Load Strength Modulii of Soils," Proceedings, First International Conference on the Structural Design of Asphalt Pavements, University of Michigan, 1962.

7.8 Seed, H. B., et al., "Prediction of Flexible Pavement Deflections from Laboratory Repeated - Load Tests, " National Cooperative Highway Research Program Report 35, Transportation Research Board, 1967.

7.9 Waters, J. M. and M. J. Shenton, "Track Foundation Design," The Railway Gazette, October 4, 1968.

7.10. Ahlvin, R. G., et al., "Structural Analysis of Flexible Airfield Pavements," Journal, Transportation Engineering, American Society of Civil Engineers, Vol. 100, No. TE3, August, 1974.

7.11 Dormon, G. M. and C. T. Metcalf, "Design Curves for Flexible Pavements Based on Layered System Theory," Highway Research Record No. 71,Transportation Research Board, 1965.

7.12 Finn, F. N., et al., "Applications of Theory in The Design of Asphalt Pavements," Proceedings, Third International Conference on the Structural Design of Asphalt Pavements, Volume I, University of Michigan, 1972.

7.13 "Full Depth Asphalt Pavements For Air Carrier Airports," The Asphalt Institute, Manual Series No. 11, January, 1973.

7.14 Deen, R. C., et al., "Structural Analysis of Bituminous Concrete Pavements," Highway Research Record, No. 407, Transportation Research Board, 1972.

7.15 Jung, F. W., and W. A. Phang, "Elastic Layer Analysis Related to Performance in Flexible Pavement Design," Research and Development Division, Ontario Ministry of Transportation and Communications, Report Number RR191, March, 1974.

7.16 Prause, R. H., et al., "Assessment of Design Tools and Criteria for Urban Rail Track Structures - Volume I, At-Grade Tie-Ballast Track," Urban Mass Transportation Administration, Department of Transportation, Report No. UMTA-MA-06-0025-74-3, April, 1974.

7.17 "Construction and Maintenance Operations," Proceedings, American Railway Engineering Association, Volume 75, Bulletin 645, November-December, 1973.

7.18 Ireland, H. O., "Railroad Subgrade Stresses," Proceedings, American Railway Engineering Association, Bulletin 641, January-February, 1973.

7.19 Talbot, A. N., "Stresses in Railroad Track," Reports of the Special Committee to Report on Stresses in Railroad Track.
- First Progress Report, Proceedings, AREA, Vol. 19, 1918, pp. 873-1062.
- Second Progress Report, Proceedings, AREA, Vol. 21, 1920, pp. 645-814.
- Third Progress Report, Proceedings, AREA, Vol. 24, 1923, pp. 297-453.
- Fourth Progress Report, Proceedings, AREA, Vol. 26, 1925, pp. 1081-1245.
- Fifth Progress Report, Proceedings, AREA, Vol. 31, 1930, pp. 69-336.
- Sixth Progress Report, Proceedings, AREA, Vol. 35, 1934, pp. 1148-1155.

7.20 Heath, D. L., et al., "Design of Conventional Rail Track Foundations," Proceedings, The Institution of Civil Engineers, London, Vol. 52, February, 1972.

7.21 Monismith, C. L., et al., "Permanent Deformation Characteristics of Subgrade Soils in Repeated Loading," A paper presented at the 54th Annual Meeting of the Transportation Research Board, Washington, D.C., January, 1975.

7.22 Barksdale, R. D., "Rutting of Pavement Materials," School of Civil Engineering, Georgia School of Technology, Atlanta, Georgia, August, 1973.

7.23 Barksdale, R. D., "Laboratory Evaluation of Rutting in Base Course materials," Proceedings, Third International Conference on the Structural Design of Asphalt Pavements, University of Michigan, 1972.

7.24 "Stresses in the Rails, the Ballast, and in the Formation Resulting from Traffic Loads," Report No. 13, Question D71, Office of Research and Experiments of the International Union of Railways, Otrecht, October, 1970.

7.25 Carey, W. M., and Irick, P. E., "The Serviceability - Performance Concept," Highway Research Board, 1960.

GPO 899-898

Technical Databases Report: Ballast and
Foundation Materials Research Program,
1975
US DOT, FRA