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Volume I: Technical Review of Ballast Compaction and Related Topics

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16. Abstract

The purpose of the research program on the mechanics of ballast compaction is to determine the influence of mechanical compaction on the ballast physical state and its consequence on the performance of the track structure. This report, which is one of a series for this project, presents the results of an extensive literature review on ballast compaction and related topics. The topics covered are the track system components, aggregate material characterization, mechanics of granular materials, compaction of granular materials, present practice of ballast related track construction and maintenance, relationship of ballast compaction to track performance, assessment of effects of compactor parameters, and economics of track maintenance.

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PREFACE

This study was conducted at the State University of New York at Buffalo (SUNYAB) for the Transportation Systems Center of the U. S. Department of Transportation in Cambridge, Massachusetts, supported by funds from the Federal Railroad Administration. This report is a critique of available information on analytical techniques, experimental laboratory and field investigations, and the economic considerations important to the behavior of compacted ballast materials. The work is part of a contract to evaluate ballast compaction and recommend guidelines for using compaction to improve track performance. The technical monitor was Andrew Sluz.

The Principal Investigator for the study was Ernest T. Selig, Professor of Civil Engineering at SUNYAB. Compilation of the material described in this report was principally provided by Tai-Sung Yoo, Research Assistant Professor, and Carmen M. Panuccio, Research Engineer.

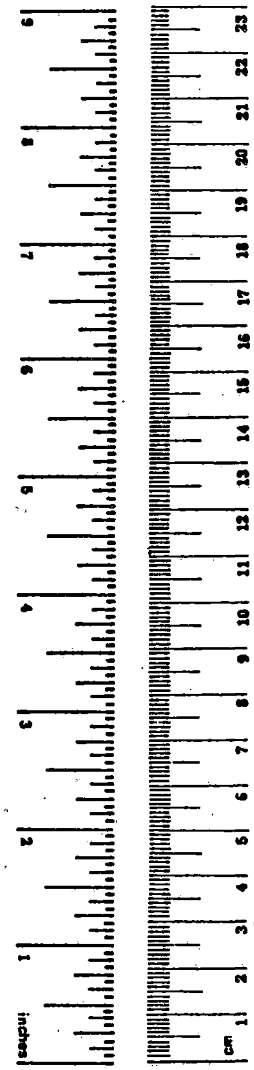
The authors would like to extend their appreciation to 1) David Burns, Railroad Industrial Engineering Consultant and former Illinois Central Gulf Railroad Cost Consultant, 2) F. L. Peckover, Railroad Geotechnical Consultant and former Geotechnical Engineer for the Canadian National Railways, 3) Stephen Brown, Senior Lecturer in Civil Engineering, University of Nottingham, England, and 4) Warren B. Peterson, Assistant Chief Engineer, Maintenance of Way, for the Soo Line Railroad, each of whom reviewed the manuscript and offered valuable suggestions. Mr. Burns also added much of the information in Chapter IX on Economics of Track Maintenance.

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

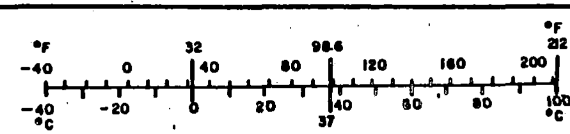
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2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

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



Approximate Conversions from Metric Measures

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



ADDITIONAL CONVERSION FACTORS

		To Convert			
Units	Symbol	From	To	Multiply By	Symbol
length	in.	inches	millimeters	25.4	mm
	ft	feet	meters	0.305	m
	miles	miles	kilometers	1.61	km
	yd	yards	meters	0.914	m
area	sq in. ($in.^2$)	square inches	square centimeters	6.45	cm^2
	sq ft (ft^2)	square feet	square meters	0.0929	m^2
force	lb	pounds	newtons	4.45	N
	kip	kips	kilonewtons	4.45	kN
	ton	ton(short)	ton (metric)	0.907	t
	kg	kilogram	newtons	9.81	N
	kp	kilopond	newtons	9.81	N
	Mp	megapond	kilonewtons	9.81	kN
pressure	lb/sq in. (psi)	pounds per square inch	kilonewtons per square meter	6.89	kN/m^2
	kg/cm^2	kilogram per square centimeter	kilonewtons per square meter	98.07	kN/m^2
volume	cu ft (ft^2)	cubic feet	cubic meters	0.03	m^3
	cu yd (yd^3)	cubic yard	cubic meter	0.765	m^3
density	lb/ft^3 (pcf)	pounds per cubic feet	megagrams per cubic meter	0.016	Mg/m^3

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LIST OF ABBREVIATIONS AND SYMBOLS

A = area (in.^{2})

A_m = maximum acceleration (in./sec.^{2})

AAR = Association of American Railroads

AASHTO = American Association of State Highway and Transportation Officials

AREA = American Railway Engineers Association

ASTM = American Society for Testing and Materials

B = width of soil particle (Chapter 3)

B = particle breakage factor (Chapter 4)

B = annual traffic density (MGT) (Chapter 9)

BBI = Ballast Bearing Index (lb/sq in.)

CBR = California Bearing Ratio

CNR, CN = Canadian National Railways

CPR = Canadian Pacific Railways

CWR = continuously welded rail

D, d = particle diameter (Chapter 3)

D = total annual tonnage or density (MGT) (Chapter 9)

D_{ave} = average particle size of the replaced fraction

D_e = equivalent particle diameter

D_{ijk} = annual tonnage carried in type i wheel loads, at class j speeds, in train service type k

D_{max}, d_{max} = maximum particle diameter

d_{ave} = average particle size of the replacement fraction

E = particle elongation

E_r = Resilient Modulus (lb/in.^{2})

e = void ratio

e = base of natural logarithms

F = particle flatness

F_{ijk} = multiplier which relates the effect upon rail life or surfacing cycle of type i wheel loads carried at class j speeds, in train service type k

FAST = Facility for Accelerated Service Testing

f = frequency (Hz)

G_p = packing specific gravity

G_s = apparent specific gravity or specific gravity of the replaced fraction

g = acceleration due to gravity (in./sec.^{2})

g_s = specific gravity of the replacement fraction

H = thickness of soil particle (Chapter 3)

H = tie life (years) (Chapter 9)

I, I_x = moment of inertia (in.^{4})

I_{90} = moment of inertia for 90 lb rail (in.^{4})

J = surfacing cycle length (years)

K = relative life factor which reflects the physical condition associated with the track structure

K_o = coefficient of at-rest lateral pressure

k = modulus of subgrade reaction (lb/in.^{3})

L = length of soil particle (Chapter 3)

L = relative life factor which reflects the influence which tie plate size, rail type (CWR, jointed), gradient and track support quality have upon tie life (Chapter 9)

M = relative cycle length factor which relates the effects of gradient, track support quality, and rail weight on surfacing cycle length

M_c = annual track maintenance cost per mile of single track

MGT = million gross tons

mph = miles per hour

N = number of the corners of edges of a particle

n = an index varying from 0.15 to 0.55

P = percentage of the replaced fraction by weight

p = percent passing a given d by weight, or percentage of the replacement fraction by weight

Q = rail seat load (lb)

R = radius of the maximum inscribed circle within the particle

R_i = individual radius of a corner or edge of a particle

R_o = particle roundness or angularity

S_o = degree of saturation (%)

SPTC = Southern Pacific Transportation Co.

T = total main line track rail life (MGT)

t = vibration time (sec.)

U = pore water pressure (Chapter 4)

U = undisturbed state (Chapter 7)

U_f = pore pressure at failure

V = volume of soil particle (Chapter 3)

V = train speed (mph) (Chapter 9)

W = weight of sample (Chapter 7)

W = weight of rail in pounds per year (Chapter 9)

X = particle sphericity

Δ = horizontal deformation

ΔV = volumetric strain (%)

ε, ε_1 = axial strain (%)

γ = sample density (pcf)

γ_s = density of the solid particles

γ_w = density of water

η = porosity

ϕ = angle of shearing resistance

ϕ_u = undrained angle of shearing resistance

σ = stress

σ_c = confining pressure

σ_d, $\Lambda\sigma$ = change in stress

σ_n = normal stress on failure plane

σ_1 = total axial stress

σ_2 = orthogonal lateral stress to σ_3

σ_3 = total lateral confining stress

$\bar{\sigma}_3$ = effective lateral confining stress

$(\sigma_1 - \sigma_3)$ = axial compressive stress

$(\sigma_1 - \sigma_1)_f$ = axial compressive stress at failure

τ = shearing strength

ω = water content (%)

EXECUTIVE SUMMARY

This report provides a technical review of literature concerning ballast compaction and ballast-related factors influencing track performance.

The performance of railroad track systems is a function of the characteristics and the complex interactions of the track system components under traffic and environmentally-induced stresses. Descriptions of the track components are presented in the report, including the rails, ties and fasteners, and in particular, the ballast, subballast and subgrade. Also, the relationships which exist between the track system components and track performance have been discussed. The ballast and subgrade behavior were shown to have a significant effect on the performance of the track in-service.

Recently developed analytical track models have been described with particular emphasis concentrated on the representation of the ballast and subgrade materials. However, the reliability of these models for predicting track performance was not established. The primary reason is the lack of corroboration with field data. Available field measurements or criteria that are indicators of track performance are track stiffness, track geometry, safety, ride quality, and maintenance effort. However, each individual item is not sufficient for proper representation of the overall track system performance.

Ballast materials possess different particle physical and chemical properties. Many laboratory index property tests, such as those for abrasion resistance, absorption, shape and soundness, are currently utilized to quantify and categorize the relative merits of these different ballast types. The applicable test standards and ballast specifications limits have been cited and the basic test procedures, as well as the factors influencing the test results, have been discussed. However, at present, a proven method for rating ballast using these index properties does not exist.

The stress-strain behavior of ballast-sized materials may be determined from laboratory tests. The conventional static and dynamic test apparatus and procedures, as well as the factors affecting the test results, have been discussed and assessed with respect to use with ballast materials. The cyclic or repeated load triaxial test appears to be particularly suitable for determining the ballast stress-strain and strength properties. Laboratory test data need to be correlated with in-situ data representing these properties. An evaluation of the available field methods indicates that the plate bearing test is best suited for this purpose.

The strength and compressibility characteristics of ballast are directly related to the relative degree of compaction of ballast within the track structure. Methods used in geotechnical engineering practice for measuring and specifying compaction are reviewed. However, it was shown that these methods have limited application for ballast materials. Furthermore, no quantitative ballast compaction specifications are available and measurement of the degree of ballast compaction is rarely done. However, an in-situ ballast density test, a small plate load test, and a single lateral tie push test on an unloaded tie have been identified as potential means of representing the ballast physical state.

The rate of change in the ballast physical state within the track structure caused by train traffic loading and environmental conditions is highly dependent upon the initial state achieved from the track maintenance operations. Ideally, an "undisturbed" ballast trackbed after considerable traffic is the most desirable condition from a stability viewpoint. The reason is that the subsequent correction of track geometry defects during maintenance by ballast tamping disturbs the stable condition created by traffic. Therefore, the effect that present track maintenance processes have upon changing the ballast physical state should be studied. To aid in understanding these effects, the principal types of track maintenance equipment and the associated field procedures in current use have been described.

The ballast state produced by the tamping, leveling and lining operations, and the more recent ballast crib and shoulder compaction process after tamping are discussed in relation to their expected effect on track performance under traffic following maintenance. Lateral resistance of either single tie or tie panel sections is the most frequently used method for measuring track performance related to ballast conditions. However, alternative methods, including track geometry changes, track or ballast stiffness, and ballast density have also been used. Supporting evidence obtained with these methods and published opinion indicate that mechanical ballast compaction following the tamping operations should be beneficial. The degree and nature of the benefit was not clearly established, however.

The amount that the ballast is compacted with present crib and shoulder compactors is a complex function not only of the initial ballast physical state, but also the compactor characteristics including static force, generated dynamic force, vibration frequency, and duration of vibration. However, sufficient information is not available to determine the effects of these factors on ballast compaction. Therefore, laboratory and field investigations are required to find the most effective means of obtaining the desired physical state of ballast.

The economic aspects of track maintenance are briefly discussed to provide a basis for assessing the cost-effectiveness of ballast compaction. This subject was shown to be complex and requires further study.

The practices and principles of geotechnical engineering provide direct and valuable input into understanding the behavior of the ballast, subballast and subgrade materials under the imposed loading environment. However, the lack of field measurements and the incomplete development of both laboratory property tests and computer track design models require further research in order to establish the needed understanding of ballast properties and their relation to track performance.

1. INTRODUCTION

This study basically concerns ballast compaction and its relationship to track performance. As part of the study, an extensive review was made of a wide range of related topics. Some of these topics deal with geotechnical engineering and others with railroad engineering. One of the purposes of this report is to relate these two disciplines. It is expected that some of the basic information will be well known to some readers, but unfamiliar to others. The report should be a convenient reference, either as a refresher or an introduction to the topics covered.

Section 2 of the report generally describes the current design practices and the various components used in the construction of conventional track structures. Emphasis on the substructure (ballast, subballast, and subgrade) responses as determined from analytical models and observed field measurements will be discussed in relation to track performance. The chemical and physical properties used to characterize ballast materials are presented in Section 3 along with the appropriate test specifications and typical values. Section 4 is principally concerned with the behavior of granular materials subjected to static and dynamic loading in laboratory strength property tests and in field tests. The physical state of granular materials, which is usually expressed in terms of density, that is achieved by laboratory methods and by conventional field compaction equipment is described in Section 5. Included are several proposed mechanisms explaining the compaction process. This information is directly relevant to the ballast physical state conditions achieved by current track maintenance equipment and practices, especially the tamping-leveling-lining and ballast compaction operations, presented in Section 6. The effect that track maintenance operations, in particular the addition of ballast crib and shoulder compaction, has upon track performance is discussed in Section 7. The important parameters affecting the compaction of ballast materials are more thoroughly assessed in Section 8. Finally, a general treatment on track maintenance costs is contained in Section 9 to establish the basis for potential economic relationships between ballast compaction and track performance.

2. TRACK SYSTEM

The purpose of this chapter is to provide a brief review of the track system components and performance. This necessary background information for the research is presented primarily for those readers not already familiar with the subject. In addition to original source documents, a technical report prepared by the University of Illinois (Ref. 1) was particularly useful in providing information for this chapter.

2.1 DESCRIPTION OF TRACK COMPONENTS

The three main parts of the conventional track system are the track structure, the ballast (and subballast) and the roadway. The components of the track structure are the rail, the tie and the tie-plate, the fastenings, and auxiliary facilities such as signal and communication systems. In addition to the subgrade supporting the ballast and the track structure, the roadway includes the drainage system for the track. All of these components play a significant role in the performance of the system under various loading conditions and environmental changes. However, the ballast and the subgrade are the most complex and least understood part of the system.

The track components described in this chapter are used in all track systems. However, the specific details vary from one country or state to another. Emphasis in this report is on American and Canadian practice.

Rail. The rail is the immediate structure supporting and guiding the rolling stock. It reduces the level of loads from the train, and distributes them over the ties. The size and cross-section of the rail have been subjected to continuing changes over the years. Consequently, a variety of rail designs with different weights, ranging from 60 to over 150 lb/yd, have been available in the market. However, AREA now recommends only the six standard sections listed in Table 2.1 (Ref. 2).

Selection of rail size depends on such factors as the traffic, the structural requirements, availability of rail, and the economics of track

Table 2.1. AREA Recommended Rail Sections (Ref. 2)

Rail Size	Weight lb/yd (Kg/m)	Area, A in.^{2} (cm^2)	Moment of Inertia, I_x in.^{4} (cm^4)
90 RA-A	90.0 (44.7)	8.82 (56.90)	38.7 (1611)
100 RE	101.5 (50.4)	9.95 (64.19)	49.0 (2039)
115 RE	114.7 (56.9)	11.25 (72.58)	65.6 (2730)
132 RE	132.1 (65.5)	12.95 (83.55)	88.2 (3671)
136 RE	136.2 (67.6)	13.35 (86.13)	94.9 (3950)
140 RE	140.6 (69.8)	13.80 (89.03)	96.8 (4029)

construction and maintenance. There is no rational method considering the overall aspects on the above requirements. Instead, the usual procedures are to establish minimum size based on the structural and electrical needs, and then to consider the economic aspects for the final selection of the rail. The recent trend of rapidly increasing labor costs has created a general tendency to use heavier rail than required by the structural needs to reduce the cost of maintenance and replacement work.

Conventional rails are joined together with bolted joint bars that connect rails of standard lengths. Rail lengths of 39 ft have been a recent standard, but to halve the number of joints, rail welded into 78 ft lengths is now commonly used on main lines.

The bolted rail joints have been one of the major locations of maintenance problems. Discontinuity of track running surface produces dynamic impact loads battering the rail surface and the joint ends. This creates rough riding track and undesirable train vibration. The combination of the impact load and the reduced rail stiffness at the joints causes greater stress on the ballast and subgrade. This in turn, increases the permanent settlement which produces uneven track. The pumping action at the joints also accelerates rail failure, tie wear, and fouling of ballast at the joint. Hence joints generally increase track deterioration.

Although much progress has been made in improving joints, a better solution is to eliminate the joint entirely by the use of continuously welded rail (CWR). Various advantages of CWR include substantial savings through reduced maintenance costs through extended rail life from the elimination of joint wear and batter, improved riding quality, reduced wear and tear on rolling stock, and less deterioration of ballast and sub-grade conditions. Disadvantages are 1) breakage or buckling of track from temperature induced changes in rail stress, 2) difficulties in changing worn and defective rails, or in tie renewal, and 3) higher initial costs of welding, transporting, and laying longer rails.

Tie and Tie-Plate. The tie, or more specifically the cross tie, receives the load from the rail and distributes it over the underlying ballast. The tie also plays the important role of holding the two rails at the correct separation or gage. The tie is usually a piece of timber varying in cross section from 6 in. by 6 in. to 7 in. by 9 in. and in length from 8 to 9 ft (Ref. 1). However, with diminishing availability and increasing price of wood, alternative forms of ties have been developed, the most common of which are concrete ties. Longer life is one expected advantage of the concrete ties. In addition, the greater weight of concrete ties provides an important stability advantage over wood with continuously welded rails.

A steel tie-plate is inserted between the rail base and the top of the tie to protect the tie from mechanical wear, and to distribute the rail loads throughout the bearing area of the plate, therefore reducing the maximum pressures on the tie. The shoulder of a tie-plate, when secured firmly onto the tie, help restrict the lateral motion of the rail to hold proper line and gage. Various designs and sizes of tie-plate are now available, generally ranging from a width of 7-3/4 to 8 in., and a length from 12 to 14 in. The size of the tie-plate is an important factor in determining the stress reduction in ties; however, selection of the plate is usually based on economic considerations. Commonly, a plate which gives the least overall cost for the tie and plate combined is used.

Fastening and Anchoring. The fastening is the key element holding the rail to the tie or other support. Generally speaking, the basic role of the fastening is to maintain the rail gage, and to restrain the lateral and longitudinal movements of the rail. However, the effectiveness of this role varies with the type of fastener and the type of the track structure. In the conventional track with ballast and wooden ties, the role of the fasteners, which are usually cut or screw-type steel spikes driven directly against the edges of the rail base, is essentially limited to maintenance of the track gage. The spikes driven into wooden ties are not expected to hold the track structure vertically under traffic or prevent longitudinal rail movement. However, in the case of concrete ties, the fastener maintains not only lateral alignment, but also provides vertical and longitudinal restraints to the rail movement. In such cases, the resiliency of the fastener becomes an important performance factor.

The major role of the anchor is to transmit to the ties the longitudinal rail loads generated by the acceleration and braking of the trains and by the thermal expansion and contraction forces. For the jointed rails, it also helps to maintain proper expansion allowance at the joint gaps, thus assisting in maintenance of proper line and gage. The number of anchors needed increases when the ballast in the cribs is not properly compacted.

Ballast and Subgrade. The ballast is selected noncohesive material placed on top of the track subgrade to support the track structure. Conventional ballast is the uniformly-graded and angular granular aggregate which is tamped under and around the ties, and performs several important functions:

1. to limit tie movement by resisting vertical, lateral and longitudinal forces from the train and track.
2. to reduce the stresses from train loads applied to the subgrade of roadbed, thus limiting permanent settlement.
3. to provide immediate water drainage from the track structure.
4. to help alleviate frost problems.
5. to facilitate maintenance surfacing and lining operations.

6. to retard vegetation and resist effects of fouling from surface deposited materials.

7. to provide support for ties with the necessary resilience to absorb shock from dynamic loads.

Traditionally, angular, crushed, hard stones and rocks, uniformly graded to drain freely, free of dust and dirt, and not prone to cementing action have been considered good ballast materials. However, at present no universal agreement exists concerning the specifications for the combination of the ballast material index characteristics such as size, shape, hardness, abrasion resistance, and composition that will provide the best track performance. This is a complex subject that is still being researched. Availability and economic considerations have been the prime factors considered in the selection of ballast materials. Thus, a wide variety of materials have been used for ballast in the United States and Canada such as crushed granite, basalt, limestone, slag and gravel (Ref. 3). The best materials, such as crushed rock, are used on the main line track, while the poorest, like gravel, are usually restricted to use on sidings or spur lines.

AREA specifications (Ref. 2) recommend five different gradations for crushed stone and crushed slag. Most of the gradations fall within the size range of about 1/4 in. to 3 in. According to Ref. 3 published in 1957, the most commonly used ballast gradations were AREA No. 4 (nominal size range = 3/4 in. to 1-1/2 in.) and AREA No. 3 (nominal size range = 1 in. to 2 in.) AREA also recommends three different gradations for gravel (Ref. 2), depending on the percentage of crushed particles specified. The particle size range varies from 1-1/2 in. down to approximately medium sand size (about 0.4 mm). The FRA specifications for pit-run gravel ballast (Ref. 4) extend the upper end of the size range to 2-1/2 in.

The mechanical properties of ballast result from a combination of the physical properties of the individual ballast material and its in-situ (i.e., in-place) physical state. Physical state can be defined by the in-place density, while the physical properties of the material can be described by various indices of particle size, distribution, shape, angularity and hardness. The in-place density of ballast is a result of some type of compaction process. The initial density is usually created by maintenance tamping, and subsequent density changes result from train traffic combined with environmental factors. Experience has shown that tamping does not produce a high degree of compaction and there is clearly little geometry control in achieving compaction by train traffic. Therefore, consideration is now being given to additional compaction during maintenance using special machines and/or new techniques.

Subballast is used as a transition layer between the ballast and the subgrade. In most new construction, the subballast prevents the mutual penetration or intermixing of the subgrade and ballast and reduces frost penetration into the subgrade, in addition to fulfilling some of the functions of the ballast. Subballast thus reduces the required thickness of ballast, usually a more expensive material, thereby providing an economic benefit.

Any free-draining sand and/or gravel materials could serve as the subballast, as long as they meet the proper requirements of a filtering material. AREA specifies that materials for subballast should conform to ASTM D-1241, "Standard Specification for Soil-Aggregate Subbase, Base, and Surface Courses." The thickness of the subballast to be placed on the completed subgrade may vary, but usually is specified as 12 inches or less. Where practical, subballast should be placed in layers and thoroughly compacted in accordance with standard practice so as to form a stable foundation for the ballast.

The subgrade is the layer of material on which the ballast/subballast layers rest, whose functional requirements are to:

1) support, without appreciable permanent deformation, the maximum dynamic, traffic-induced stresses transmitted through the ballast.

2) resist the cyclic stresses without excessive cumulative volume or strength reduction.

3) be non-frost susceptible and be volumetrically stable during cycles of wetting and drying.

4) resist softening that causes pumping and penetration into the ballast.

The subgrade is a very important component in the track structure, which has frequently been the cause of track failure and the development of poor track. Unfortunately in existing track, the subgrade is not involved in the maintenance operation and little can be done to alter its characteristics without major track reconstruction, i.e., removal and replacement of track, ballast and subballast.

The present state-of-the art of track design as it concerns the ballast and subgrade is very empirical, and the factors controlling performance are poorly understood. Reliance on past experience can be very misleading, because not only is the experience at a particular site a complex and unknown function of many factors, but the controlling factors are often not even adequately documented. For example, to assess the reasons why a particular section of track might be "poor track", it is necessary to know 1) the characteristics of the ballast and subgrade, 2) the maintenance history including frequency and type of operation, 3) environmental factors, and 4) traffic history. Only the last item is readily determined, although the second and third can sometimes be estimated from available records. Necessary information of the characteristics of the ballast and subgrade of existing track, however, is practically non-existent. Even the classification of these materials is in doubt, not to mention their physical state. At best, a knowledge of the present conditions of a site based on a field examination is all that is possible, because past records are not normally available.

2.2 TRACK SYSTEM RESPONSE

Loading. While performing its functions of supporting and guiding the rolling wheels, the track system receives various loadings from the trains and the track structure. In addition, the track system is also subjected to expansion and contraction loads from environmental changes, especially temperature.

The nature of the loading condition of a track system is quite complex, and its magnitude varies with the characteristics of the whole system and the interaction between each component. However, it may be conveniently categorized into the three orthogonal components as follows:

1. Vertical loads from a) the static weight of the track, and from b) the dynamic forces generated from the train motion.
2. Lateral loads from a) the train reaction to the track geometry deviations, such as forces from self-excited hunting motions, b) the forces necessary to guide the train through curves, and c) resistance to thermal expansion or "sunkinks."
3. Longitudinal forces from a) traction and braking of the trains, and b) thermal expansion and contraction of the rails, especially when continuously welded rails are used.

The vertical loading conditions have been given the most attention in the past. Gross vehicle loads range upward to about 130 tons (120 metric tons) for some cars, to 140 tons (130 metric tons) for locomotives. Wheel loads obviously depend on the number of axles per vehicle (usually 4 for cars and 4 to 6 for locomotives), the distribution of load, and the gross load. The upper limit on static wheel loads is on the order of 25,000 to 35,000 lb (11,300 to 15,900 kg.)

Dynamic or impact loading caused by train in motion can be substantially greater than the static values. The dynamic effects have not been well defined, but estimates have ranged from 50 to 100% of the static loads, to a one percent increase for each 1 mph speed increase over 5 mph (Ref. 5, 6). Furthermore, severe rock and roll can sometimes produce wheel lifting from a rail, causing up to double the wheel load on the adjacent rail.

Main line track traffic volumes range from one million or less gross tons per year to about 50 to 60 million gross tons (45 to 54 million metric tons) annually. Assuming an average axle load of 30 tons, this represents 33,000 to 2,000,000 cycles of load application.

Lateral wheel loading characteristics are very complex and much more difficult to define. Instead, the maximum lateral forces are usually derived as a percentage of the vertical load, which is known as the derailment quotient. A derailment quotient of about 0.8 has been suggested by Prause et al. (Ref. 5) for estimation of the maximum expected lateral force based on the nominal static wheel load. Lateral forces caused by "sunkinking" are also not well defined. An empirical value of 300 lb lateral restraint per tie in an unload condition has been suggested by Magee. (Ref. 7)

<u>Analytical Track Models.</u> The principal function of a track model is to interrelate the components of the track structure to properly represent their complex interaction in determining the net effect of the traffic loads on the stresses, strains and deformations of the system. Such modeling provides the foundation for predicting track performance, and therefore technical and

economical feasibility of track design and maintenance procedures. Analyses are complicated, however, by the fact that the physical states of the ballast and subgrade, but especially the ballast, change with time. Because maintenance life is measured in years, these long-term effects must be considered. A considerable amount of effort has been devoted to the development of track models that could realistically represent the actual behavior of the track system subjected to various loading conditions. However, more research is needed for reasons including: 1) the difficulties of handling the complexities inherent to each component of the track structure and their interaction under loads, 2) lack of adequate understanding on the ballast and subgrade behavior to define the model requirements, 3) lack of field data on track performance for validating the models, or 4) high computer costs in running the most elaborate of the computer models.

Since the railroad track is generally subjected to three-dimensional loads, i.e., loads in vertical, lateral, and longitudinal directions, various analytical models have been suggested for each of these components of the track response or for multi-dimensional representations. However, the vertical behavior of the track structure has received the major effort. The following is a brief summary of the existing models which are available for vertical response analysis of conventional railroad track.

Based on a theory of a continuous beam on an elastic foundation, Talbot (Ref. 6) made significant contributions in understanding the behavior of a railway track system under vehicle loading. The concept of "track foundation modulus" was introduced, and mathematical formulations were developed for calculation of the deflection and moment in the rail. Clarke (Refs. 8, 9) summarized the above approach to present a basis for track design procedures. However, this theory does not include several important factors which are known to affect the stresses and deflections in railroad track, such as longitudinal loads from thermal stresses, a restoring moment proportional to the rotation of the rail and ties, the eccentricity of the vertical load on the rail head, or any track dynamic effects. In addition, a rather significant limitation to the approach is that it does not adequately model the stress-strain behavior of the ballast and subgrade.

Meacham, et al. (Refs. 10, 11) and Prause, et al. (Ref. 5) attempted to overcome some of the limitations of the earlier beam on elastic foundation approaches by developing a theoretical method for the determination of the "track modulus" value. Each component of track structure was represented by a series of elastic springs. The spring stiffness was computed by considering various track parameters, such as rail type, tie type, ballast depth, ballast type, subgrade type, and tie-spacing.

The finite beam on elastic foundation approach is basically similar to the above theories, except that it considers the tie as a finite beam resting on an elastic (Winkler-type) foundation to represent the response of a tie resting on the ballast. The approach was extensively studied by Hetenyi (Ref. 12), and various analysis methods for the solution have been presented. For example, Barden (Ref. 13) considered non-uniform foundation modulus, and Harrison, et al., (Ref. 14) included a non-uniform beam section and a non-uniform foundation as well. An approximate analytical method was developed which makes assumptions about the distribution of wheel load over the rail and across the ties. The vertical stress distribution with depth in the ballast and subgrade layers under any given tie is then computed using the Boussinesq theory. Ireland (Ref. 15) presented a design chart for ballast-subballast depth selection versus cohesive strength of subgrade soil using this approach.

An approach was developed at AAR that uses Burmister's multilayer theory for the ballast and subgrade and a structural model for the rail-tie interaction. The contact between a tie and the ballast was represented by a series of circular areas with uniform pressure. The superstructure and the substructure models were combined and extended by Battelle to form the model termed MULTA (Ref. 16). This is a three-dimensional model; however, the properties within any layer are constant and cannot be varied with horizontal position.

Finite element methods have also been applied to the track structure analysis by various researchers. Lundgren, et al., (Ref. 17) developed a two-dimensional system, assuming the plain strain behavior of a longitudinal section of unit thickness along the vertical centerline of the rail. Svec, et al., (Ref. 18) employed a three-dimensional model that represented a detailed description of the physical system. The rail-tie system was added to the model as simple beams, and non-linear mechanical properties of ballast, subballast, and subgrade were obtained from laboratory tests. One feature of the procedure was the representation of the ballast and subballast as no-tension materials. However, the model did not have clearly defined failure criteria.

Development of a finite element model, ILLITRACK, was undertaken at the University of Illinois (Ref. 19). It was not a three-dimensional model, but consisted essentially, of two two-dimensional models, one transverse, the other longitudinal, employing output from the longitudianl model as input to the transverse model. In this manner, a three-dimensional effect is obtained with less computer cost than with a three-dimensional model. Nonlinear, mechanical properties for the material were obtained in the laboratory from repeated load triaxial tests. An incremental load technique was employed to effect a solution. Explicit failure criteria were developed for the ballast, subballast and subgrade material. However, the model does not prevent tension from being transferred across the rail base into the tie plate. Further study is needed to determine whether the combined two-dimensional models employed in ILLITRACK represent three-dimensional physical conditions as expected. Certainly the three-dimensional qualities of the track structure need to be fully accounted for to successfully predict the behavior of the railway track system using finite element models.

The mathematical models developed for predicting track performance under dynamic load have been limited almost entirely to recoverable deformations, thus, they do not adequately represent the factors involved in maintenance life prediction. However, even the properties associated with recoverable deformation do not fully represent the stress-state-dependent behavior of ballast and soil under cyclic loads. Although recently a considerable effort has been devoted to studying the cyclic behavior of these materials, measures such as a resilient modulus should be designated as cyclic index properties rather than behavioral properties, because they represent only a few special stress paths and are not applicable without a factor to compensate for the effect of stress path.

Presently the approach to predicting permanent deformation of track caused by ballast and subgrade behavior is patterned after methods used in highway flexible pavement design (Ref. 20). An elastic track model is used to predict stresses in the ballast and subgrade from traffic loads, and repeated load triaxial tests are used to determine limiting threshold stress and cumulative strain as a function of confining pressure and number of cycles of deviator stress. Repeated loads start from a zero load and are increased to some predetermined magnitude and then decreased to zero, thus never putting the sample in extension in the axial direction. The process is repeated until either the desired number of cycles of a limiting permanent strain is reached. Track settlement is predicted by summing inelastic strains from the triaxial tests for the stress conditions determined from the elastic model.

Measured and Predicted Response. The nature of the recoverable deformations of ballast and subgrade as well as the stresses and strains in these materials from traffic load have been predicted using the various available track analytical models. These same response parameters have been determined experimentally on actual track structures. The resulting data have been used not only to study the track behavior, but also to evaluate the analytical models. However, the difficulty in measuring stresses and strains particularly in the ballast, has greatly restricted the amount of such data that has been obtained. The examples that follow will illustrate the general trends from both the analytical and experimental studies.

Salem (Ref. 21) studied the vertical stress distributions in the ballast and subgrade under statically loaded wood ties in a series of laboratory tests with various ballast depths, ties spacings and ballast type. Fig. 2.1 shows that chat, pit run gravel and crushed slag ballast produce nearly the same vertical pressure below the center line of a single tie. Fig. 2.2 shows the average vertical pressure distribution when 12 to 30 in. of ballast were used at a tie spacing of 21 in. Fig. 2.3 illustrates the average vertical pressure distribution on the subgrade in a longitudinal direction parallel to the tie and below its center line at a depth of 18 in. of ballast. These tests indicated that the depth of ballast section needed to get a fairly uniform pressure on the subgrade equals the tie spacing minus three inches. A comparison of measured and calculated values also indicated that, while the shape of the measured and calculated curves are similar, the calculated pressures may be considerably different from the measured data.

Analytical predictions of track response were made using MULTA for a particular range of track parameters. This analysis assumes uniform properties under the tie, which is usually not the case, and the ballast is assumed

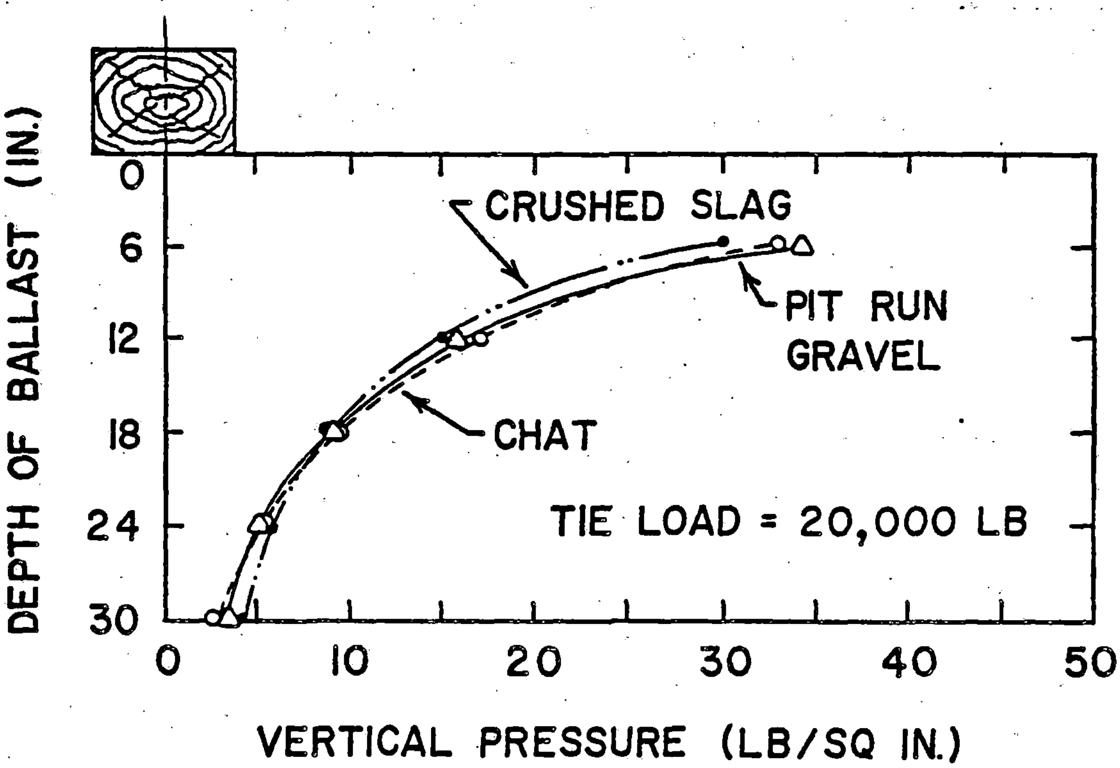

Figure 2.1. Vertical Pressure Distribution at Depths up to 30 inches of Ballast Below the Center-Line of a Single Tie (Ref. 21)

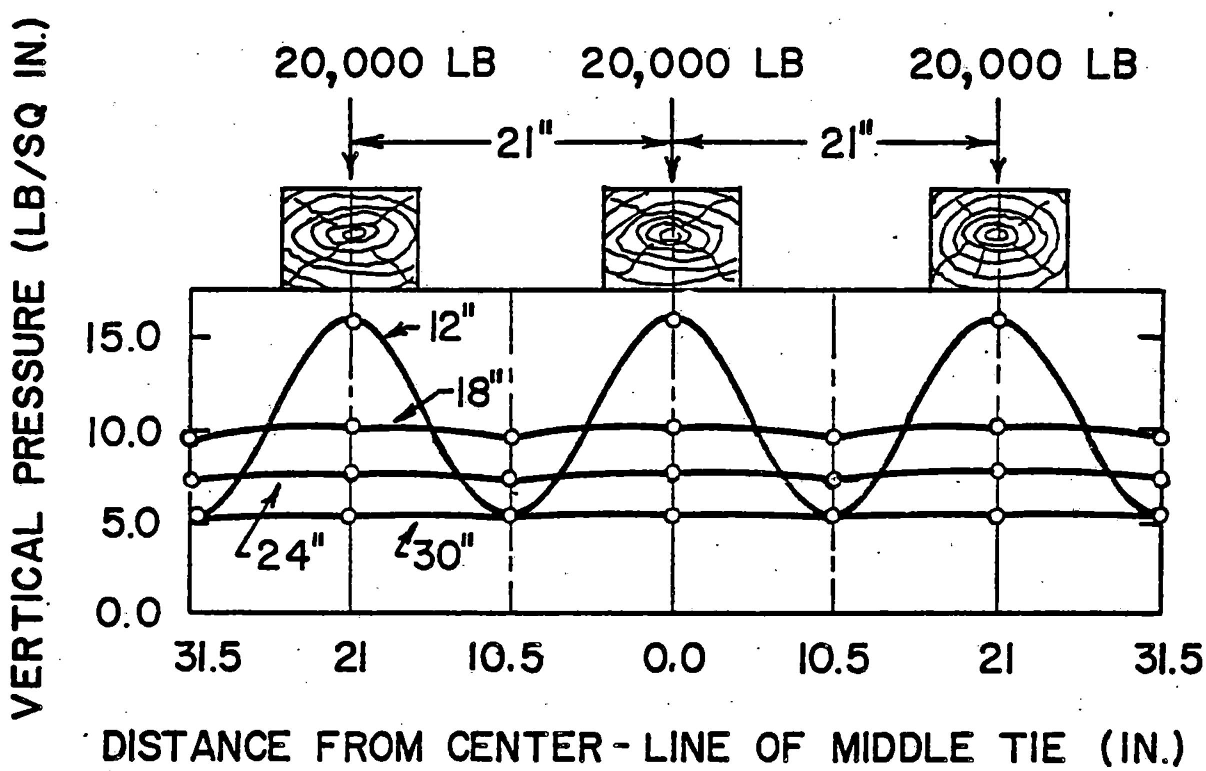

Figure 2.2. Average Vertical Pressure Distribution on the Subgrade for Different Ballast Depths (Ref. 21)

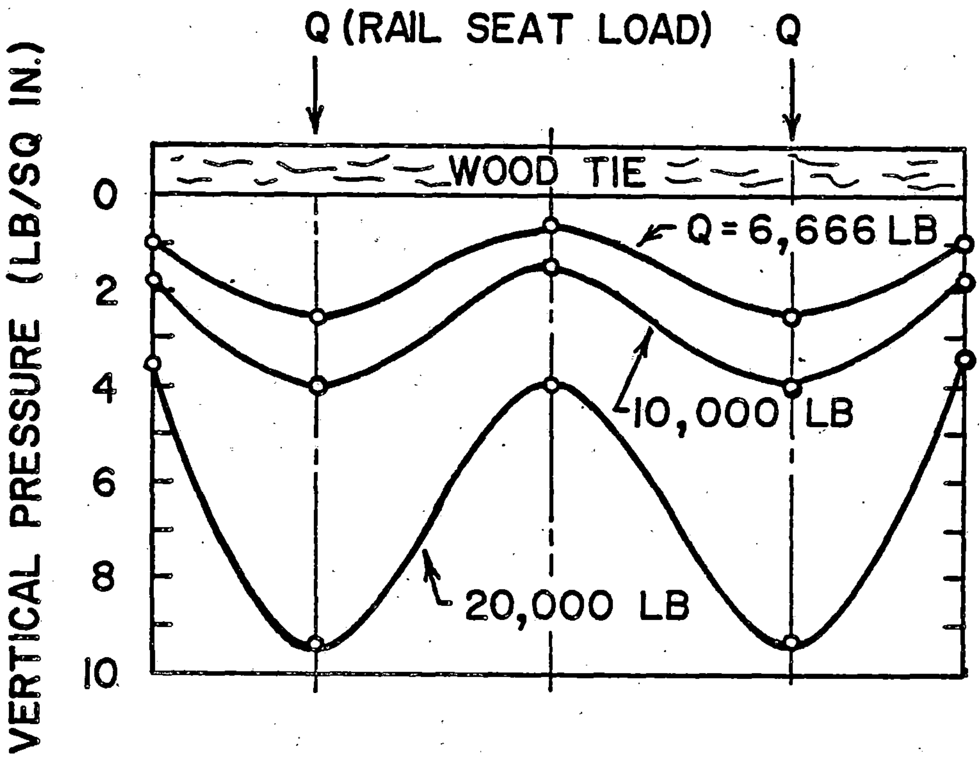

Figure 2.3. Average Vertical Pressure Distribution on the Subgrade at a Depth of 18 inches of Ballast Below a Single Tie (Ref. 21)

to be much stiffer than the subgrade. The following general trends were shown (Ref. 16):

1) The maximum bending moments at the center of the tie decrease as ballast depth increases. However, the maximum rail seat bending moments increase by a small amount, approximately 5%, when the ballast depth increases from 12 to 36 in.
2) The vertical rail displacement and the rail bending moment decrease, while the rail seat load increases as the ballast depth increases.
3) The deviatoric and bulk stresses at the mid-depth of the ballast decrease rapidly as the ballast thickness increases. However, this decrease is a result of stress attenuation with depth. Because the rail seat load and maximum pressure at the bottom of the tie increase with ballast depth increase, at a common depth in the ballast the stresses should actually increase with increase in ballast layer thickness.
4) The maximum vertical stress on the subgrade surface and the stresses in the subgrade decrease rapidly with increasing ballast thickness. This trend is also largely a result of attenuation of stress with depth.

The most extensive track response measurement program undertaken to date is being conducted at the FAST track in Pueblo, Colorado. Included are strains in the ballast and subballast, vertical stress at the subballast-subgrade interface, and vertical deformation of the subgrade surface relative to an anchor point approximately 10 ft below this surface. The strain measurement method in particular is new and provides important data not previously available. This instrumentation is described in detail in Ref. 22.

Among the observations from the records obtained when a three-car train passed slowly over the instrumented wood-tie sections are the following:

1) The permanent strain and deformation from one pass of the train was negligible compared to the elastic components.
2) The 131 ton (119 metric ton) hopper cars produced larger response than the 131 ton (119 metric ton) locomotive, because of the higher axle loads.
3) The variation in stress, strain or deformation as each individual axle in a group passes over the gage is small compared to the group average, indicating that the rail is distributing the axle loads over distances exceeding the axle spacing.
4) The vertical strain in the ballast is mostly negative (extension) beneath the center of the tie at the centerline of the track. The extension and compression strains beneath this point in the subballast are approximately equal.
5) The subgrade deflection was always downward realtive to the unloaded track position, and the subballast strains beneath the rail were essentially only compressive.
6) The ballast strains were extensional at the midpoint of the cars as a result of spring-up of the rail. However, part of this extension could be a result of lifting of the tie from the ballast because the top part of the strain gage was attached to the tie rather than to the ballast surface.

Analytical models that directly predict permanent ballast strain and cumulative track settlement from traffic loading have not been developed. Also, very little experimental data is available from the field. The current project at FAST is providing important new information on this subject, however. Cumulative ballast and subballast strain, and subgrade deflection have been measured as a function of total traffic load for a variety of track conditions. A typical set of results is shown in Fig. 2.4 for one track section. Strain measurements of this type have not previously been available. The slopes of all of these curves decrease rapidly with increasing traffic, but the permanent subgrade settlement still continues to accumulate significantly even after 100 million gross tons (91 million metric tons) loading.

2.3 PERFORMANCE AND DESIGN OF TRACK SYSTEM

Performance Measurement. The track performance is the degree of the effectiveness with which a track system fulfills its intended purpose, that of providing the rail surface conditions necessary for the safe, confortable, economical operation of the trains using the system. Each component of the track system contributes to this overall track performance. Thus track performance is also an indication of how well these components perform their individual function. Although the track performance could be illustrated in terms of the behavior of the track system components, insufficient information is available to accurately evaluate the contribution of individual components to the overall track performance.

Track performance is currently represented by a) measurements of structural capability that could be related to the track performance under loading, b) track physical appearance and its changes, and/or c) the effects of track in service, such as safety, ride quality, and derailment frequency. The most frequently used criteria for the track performance are:

1) Track Stability or Load Bearing Capacity as measured by track modulus, track settlement, or track and tie resistance.
2) Track Geometry as represented by gage, surface, twist, superelevation, or alignment.
3) Saftey as indicated by maximum allowable operating speed, slow speed restriction, or derailment frequency.

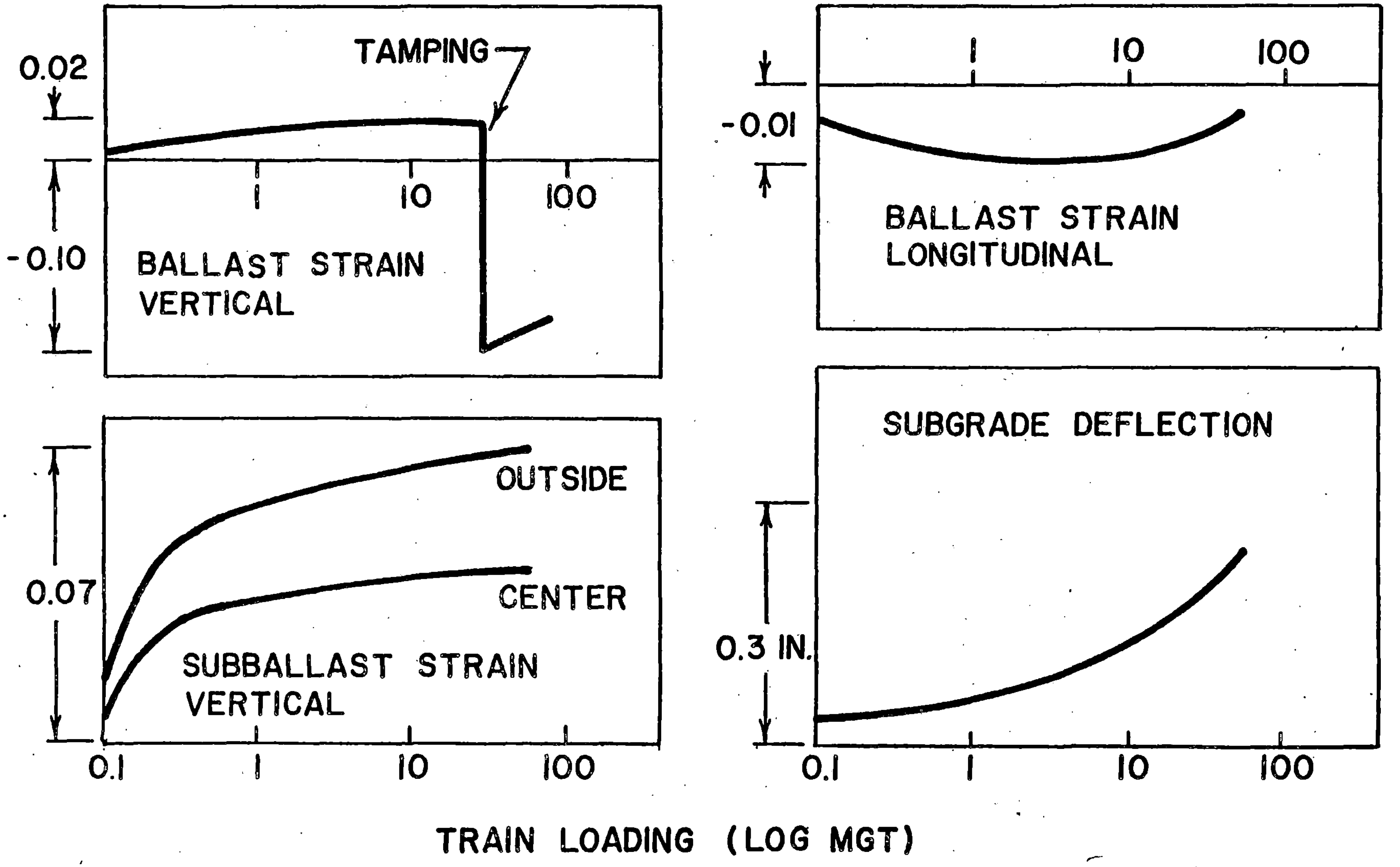

Figure 2.4. Accumulative Substructure Strain and Displacement in FAST Track Section

4) Riding Quality as represented by passenger comfort, frequency and amount of lading damage, or rate of equipment deterioration.

5) Maintenance Requirements as indicated by amount and frequency of maintenance.

6) Environmental Effects such as vibration and noise transmitted to the surrounding community.

These criteria are interrelated and none of them is universally superior to others in defining the track performance. Instead, all of these criteria ideally should be integrated into a single index measuring the relative ability of the overall track system to perform its function.

Performance can be represented either by measurement of the physical quantities representing the above listed terms, or indirectly by determination of the costs involved. The latter method could be more definitive in representing the overall picture of track performance than the former, if a reasonable scheme of converting the physical significance of the criteria into monetary values could be established. However, two major factors governing track performance which could be measured quantitatively at the present time are track geometry and track stability.

The parameters describing the track geometry are gage, line and surface. Equipment and methodology have been developed to evaluate such properties as cross level, gage, longitudinal roughness, and longitudinal differential settlement, with various rail inspection cars and/or manual procedures.

The gage is the distance measured between the inside heads of rail 5/8 in. below the top of rail. The gage must be held within reasonable limits, yet there must be some play between the wheels and rails. In the United States, according to AREA standards, this play is 19/32 to 38/32 in. Because of the normal play and the tapered tread of the wheels, lateral sinusoidal motion is normal as the train progresses. This is no problem, provided the impact speed is not too high. If the gage opens, or if the gage varies, then the force of the wheel flange against the rail will be increased, and undesirable lateral accelerations will be imparted to the vehicle body.

Line as used here refers to the adherence of the center line of the track to the established alignment as indicated by the corresponding presence or lack of irregularities and departures from the designated position at individual locations along the rails. The effects of poor alignment are rough riding, excessive and irregular rail wear, and a contribution to the development of poor surface. In multiple-track territory or other locations of limited clearance, poor alignment may also lead to dangerous clearance situations.

Surface refers to the adherence to established grade and uniformity of cross level in the plane across the heads of the two rails. It also includes adherence to the established superelevations on curves. It is important to understand that good surface applies to the track in its loaded rather than unloaded position. Track which shows perfect cross level and gage when unloaded may not be level under traffic.

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 problems such as freight damage, derailments, or equipment damage, may occur. It should be acknowledged that the tolerable levels of certain pertinent track surface conditions will vary with the users. For example, high-speed passenger train service requires different levels of track surface conditions than does a typical freight train operation. This is apparent when the FRA Track Safety Standards are examined for different classes of track.

The track stability is the capability of a track system to sustain the imposed loads from the track structure, the traffic, and the environmental changes. This stability can be measured in terms of the amounts of deformation of the track structure under load, such as in the track modulus test or the lateral tie displacement test. Details of some of these tests for indirectly measuring the track performance are described in Section 5.

Track Design. The design of the railway support system has been mostly empirical, particularly in the U.S. However, recent systems research has developed a better understanding of transient loading behavior of the railway support system. This work has already laid the basis for more rational track design methods.

Numerous theories, techniques and/or procedures have been developed for calculating stresses and deflections in the railway support system. Most, however, have concentrated on "realistic" representation of the rail-fastener-tie components, while representing the ballast and subgrade either as springs or as a linear elastic layer. Details of various techniques and procedures are summarized in Refs. 1 and 5.

The current design practice for at-grade track is based on satisfying a number of design criteria for the strength of individual track components. These criteria include the following:

1) bending stress in the rail base,
2) tie bending stress,
3) pressure on the ballast surface under a tie, or
4) pressure on the soil subgrade.

The allowable values of these criteria are briefly discussed in Ref. 1.

Although it might be expected that the rail size would depend on some type of stress criteria, other system requirements such as wear life, electrical resistance, cost and future availability, more frequently govern the rail size selection. This means that tie size and spacing and ballast thickness are the remaining design parameters which are normally varied to satisfy the component design criteria, and are the major parameters in track design tradeoff studies.

Ballast and subgrade soil materials receive inadequate consideration in the analysis and design of conventional track support structures. Strength properties of ballast and subgrade soil materials are rarely determined. Models presently used for structural analysis of the track system do not adequately consider the nature of ballast and subgrade soil materials. The ballast depth is usually selected based on its capacity to reduce the pressure from individual ties to meet an empirically-based allowable pressure limit for the subgrade. Most of the suggested design methods do not incorporate important aspects of the ballast response under the traffic, such as strength and stiffness characteristics under repeated load, and ballast property changes with climatic or environmental exposure and traffic.

Changes in Track System with Service. Non-uniform tamping, soft and decayed ties, lack of recent tamping, and depressed ballast under joints cause deterioration of surface. Over a period of time with service, irregularity in surface also develops due to differences in bearing pressures existing along the length of the ties. Usually, the repeated application of this nonuniform pressure under the ties results in a greater depression under the rails than at the center, to form a so-called "center-bound track".

Pumping joints which result from inadequate drainage, fouled ballast, and softening of the subgrade roughen the surface. Frost heave is a common cause of deterioration of the track surface during winter. The wear of all parts of the track structure also contributes to deterioration of the track surface. It becomes worsened when dynamic effects are imposed onto it. As joints wear, bolts loosen, spikes become unseated, and plate cutting and wide gage occur, the track geometry is disturbed. Geotechnical factors such as development of soft subgrade and ballast pockets, and instability of the subgrade also cause loss of surface and line. These conditions are cumulative and lead to general track deterioration if permitted to continue unchecked.

The ballast in service is subject to gradation changes caused by 1) mechanical particle degradation during construction and maintenance work, and under traffic loading, 2) chemical and weathering degradation from environmental changes, and 3) migration of fine particles. Normally, open-graded ballast is placed which facilitates maintenance operations and is free draining. If the ballast degrades, it may lose its open-graded characteristics. Additionally, in some cases cementing of the ballast may occur, which produces a layer of undesirable rigidity, reducing the resiliency.

When repeated loads are imposed on the ballast, stresses and moisture 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. A "ballast pocket" also may form when the subgrade is depressed by the stresses transmitted through the ballast. These depressions can trap water and hence lead to further track deterioration.

Saturated subgrade soils may also be pumped up into the voids of ballast by repetitive loads. Fine sand, silts and clays are susceptible to pumping when water is present. In new construction and in major upgrading to carry increasing traffic, such pumping action is reduced 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.

Subgrade pumping can be eliminated by taking corrective action such as by a) removing fouled ballast and reconstructing the track using a subballast filter layer, or b) laying a membrane under the ballast. The application of more ballast without removal of the fouled ballast and correction of the cause is not recommended, as pumping will foul the new ballast. In all cases, the total thickness of ballast and subballast must be sufficient to spread traffic loads to the subgrade without overstressing it.

If the ballast material does not possess adequate stability, repeated loading from the traffic 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, even when the ballast is properly compacted, it does not possess adequate stability for the specific conditions of loading, ballast thickness, and subgrade support, for example.

Soft spots are areas along the track where settlement requires frequent lifting to be carried out. Such spots have saturated soft subgrade and ballast pockets that trap and hold water. Heavy traffic will accelerate the problem. If track can be taken out of service temporarily, and the cost is warranted, soft spots and ballast pockets can be cured by: a) excavation and replacement of the plastic subgrade soil with more stable soil, b) stabilization of the subgrade soil, and/or c) introduction of methods to reduce the amount of water reaching the subgrade. If track cannot be taken out of service, the conditions can be improved by providing better drainage of the subgrade and/or injecting stabilizing chemicals if the soil conditions are suitable. A more reliable approach, however, is to add ballast and lay ties at a closer spacing if necessary to reduce the stresses in the subgrade material. Longer ties will also spread the traffic load over a wider area.

Frost heaving may occur in subgrade and ballast when fine-grained material and moisture exist in the track and are subjected to freezing temperature. Moist soils display volume change upon freezing, and significant volume increases occur when ice lenses develop. Rough track is caused when a difference in volume change of subgrade soil develops over short distances along or across the track.

The tamping process employed in track maintenance is generally believed to loosen ballast under the tie from its density state developed over time under traffic loading. Tamping also leaves the crib ballast very loose. Loose crib ballast is a disadvantage because it does not contribute significantly to tie lateral resistance, and it reduces the supporting capacity of the ballast under the tie by providing less lateral confinement than dense crib ballast. For this reason machines to recompact the crib and shoulder ballast after tamping are now being considered in the U. S. and Canada to speed up the process of traffic-induced-densification and provide a higher lateral track stability immediately after maintenance.

Very little direct evidence is available to support many of the above conclusions because in-situ methods of measuring the ballast physical state have been inadequate. However, new or refined methods recently developed at SUNYAB have provided tools to further study the behavior of ballast. A detailed evaluation of the techniques is given in Ref. 23.

The following factors are considered in determining needs for track maintenance:

1) Deterioration of track geometry (not included in the criteria until recent years.)

2) Failure of track structure

   a) Rail: wear, corrugation, structural failure

   b) Fastener: broken and loosened

   c) Ties: mechanical wear and decay, damage due to accident.

   d) Ballast: degradation, pumping, fouling, permanent deformation

   e) Subgrade: excessive heave, excessive and uneven settlement, infiltration of ballast

3) Reduced level of track performance

   a) Operating speed reduction

   b) Increased frequency of slow speed orders

   c) Increased rate of equipment deterioration

   d) Increased lading damage

   e) Deterioration of riding quality

   f) Increased level of noise and vibration

   g) Increased detailments and accidents

   h) Others

4) Economical considerations

5) Availability of maintenance

The above factors are interrelated one to another. It is not known which ones receive the most consideration, and what the criteria are for each factor.

3. AGGREGATE MATERIAL CHARACTERIZATION

Base courses for highways and airport pavements, shell and filter materials for earth and rockfill dams, and backfill materials for retaining structures and around culverts each use aggregates of certain type and quality which are expected to yield the required strength and other pertinent performance characteristics. However, economics is also an important factor in the material selection. Alternative types of aggregates usually are selected for a given project by subjecting representative samples to a series of index tests for preliminary ranking with respect to behavior, based on appropriate classification systems or job specifications. The types of index property tests performed on aggregates, the classification systems used as a relative ranking of different aggregate materials, and the application of these tests and systems to ballast materials will be discussed in the following sections.

Of all the engineering uses of crushed rock and gravel materials, ballast is the most severe with respect to both loading and weathering. Loads under rough rail joints are severe enough to shatter and abrade good quality rock, and exposure of the ballast bed to weathering is complete. Neither road nor concrete aggregates are subject to such severe conditions (Ref. 24).

Both physical and chemical tests are used in assessing the suitability of coarse-grained aggregates. The tests will cover such characteristics as: 1) gradation, 2) sphericity, angularity, 3) specific gravity and absorption, 4) reference density and void ratio, 5) hardness and toughness, 6) chemical soundness, 7) abrasion resistance, and 8) freeze-thaw characteristics. Such tests for ballast materials are basically for evaluating quality. Many of the quality tests have been developed as indirect indicators of potential in-service behavior. However, these quality tests quite often evaluate characteristics of the individual particles rather than the ballast mass (Ref. 1).

According to Ref. 29, most major contracts covering the selection and production of aggregates will include requirements which are based on the published standards of one or more of the following agencies:

1. American Association of State Highway Officials (Ref. 25),
2. American Society for Testing and Materials (Ref. 26),
3. Corps of Engineers (Ref. 27),
4. Federal Government (Ref. 28).

These specifications include details which are of importance to the producer as well as to the user, and they cover the range from raw material to finished aggregate. Substantially the same subjects are covered by each issuing agency, and subsequent portions of this chapter indicate the pertinent specification designations and describe the scopes of applicable specifications.

The purpose of this chapter is to summarize the tests and specifications for aggregate materials that are relevant to ballast. The review presented is a compilation of information obtained primarily from references 1 and 29, with appropriate condensation.

3.1 GRADATION

The particle size distribution dertermined by the separation of the aggregate through a progressive series of sieves stacked from the coarsest to the finest mesh screens, is known as its gradation. The cumulative percent passing each sieve is plotted as a function of the sieve or screen size representing the particle diameter. Typical gradation curves for uniform and well-graded sands and gravels are shown in Fig. 3.1. A uniformly-graded material is one with a relatively narrow range of particle sizes, while a well-graded material has a good distribution of sizes over a wide range. Gradation is the key parameter used for classification of granular materials. The grading of

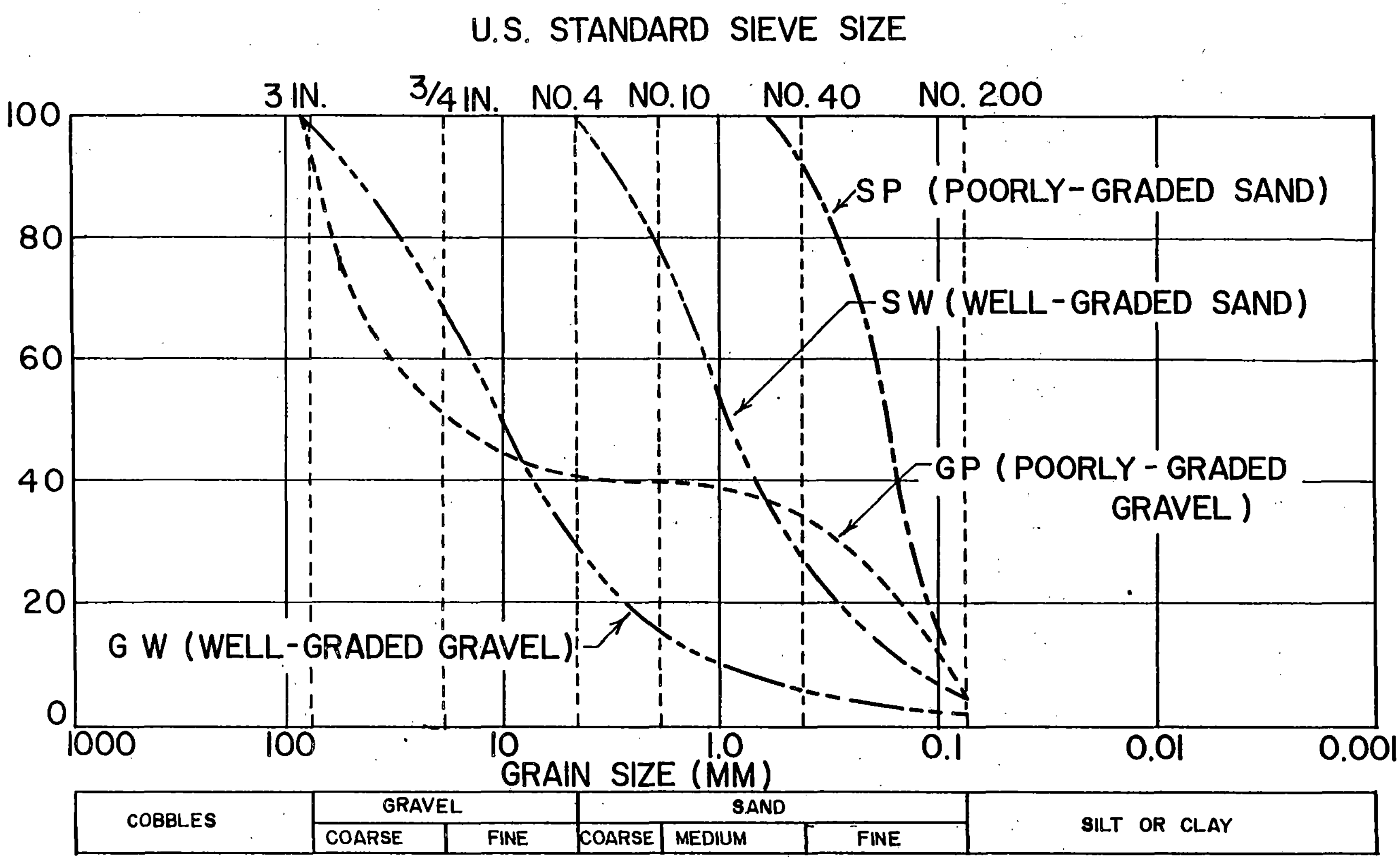

Figure 3.1. Typical Well-Graded and Poorly-Graded Soils

granular soils influences the packing characteristics, permeability, compressibility, and shear strength.

In order to reduce construction cost, a great deal of the activity of various organizations has been concentrated on the standardization and, not the least, on the uniform application of aggregate gradations. In connection with the Interstate Program, the Bureau of Public Roads made a special study of the problems involved in standardizing aggregate gradations and of the economic gains therefrom. Subsequently, in 1961, a set of specifications was issued as a guideline for the various states in connection with the federal highway projects (Ref. 30). The American Association of State Highway Officials (AASHO)*, through its committee on construction, made a complete review of all state specifications for highway construction (excluding bridges). The intent was to develop AASHO standard construction specifications which would be sufficiently broad to permit their adoption by all of the states. These AASHO Guide Specifications for Highway Construction were published in 1964 (Ref. 31).

The Bureau of Public Roads has outlined the goal of standardization of aggregate gradation specifications as follows:

1. To develop a minimum number of standard aggregate gradations that can be uniformly adopted nationwide for general usage, while at the same time recognizing the need for some variations by special provisions to fit locally available materials.

2. To achieve uniformity in the number and sizes of sieves to be used in specifying aggregate gradations.

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* Recently the name was changed to American Association of State Highway and Transportation Officials (AASHTO).

3. To develop and adopt a simple and uniform system for identification of the standard aggregate gradations (Ref. 32).

This subject was previously the basis of a cooperative study by the National Sand and Gravel Association (NSGA), the National Crushed Stone Association (NCSA), the National Slag Association (NSA), and the major producers and users of aggregate. The Joint Technical Committee proposed a set of gradation specifications, which was approved by the National Bureau of Standards and published in 1948 by the Department of Commerce as a Simplified Practice Recommendation (Ref. 33).

Standard ballast gradings are identical with standard aggregate gradings. This was decided in 1944 when the ballast gradings were first published in the AREA manual. The gradings have not been changed significantly since, in spite of the particular requirements of ballast compared to aggregates. This suggests that the ballast gradings were chosen because they were available.

The important gradation designations for highway and railroad aggregates are listed below:

1. Standard Size of Coarse Aggregate for Highway Construction:

   AASHTO Designation....................M43

   ASTM Designation......................D448

2. Crushed Stone, Crushed Slag, and Crushed Gravel for Dry-bound or Water-bound Macadam Base Courses:

   ASTM Designation......................D694

3. Crushed Stone, Crushed Slag, and Gravel for Railroad Ballast:

   Federal Specification Designation.....SS-C-743

   AREA specifications from manual (Ref. 2), the gradations for which are also given in Table 3.1.

A special study on properties of sand and gravel for railroad ballast is reported in Ref. 34.

Table 3.1. AREA Gradation Specifications for Ballast (Ref. 2)

a) Crushed Stone and Crushed Slag

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

b) Gravel

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			

c) Pit-Run Gravel

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

The standard methods of gradation determination are as follows:

1. Sieve Analysis of Fine and Coarse Aggregates:

   AASHTO Designation..............................T27

   ASTM Designation................................C136

   Corps of Engineers Designation..................CRD-C103-60

   Federal Specification Designation...............Method 202.01

2. Amount of Material Finer than No. 200 Sieve in Aggregate:

   AASHO Designation...............................T-37

   ASTM Designaiton................................C-117

   Corps of Engineers Designation..................CRD-C105-57

   Federal Specification Designation...............Method 202.11

AREA specifies that the amount of material in ballast finer than No. 200 sieve be limited to a maximum of 1%.

3.2 SPHERICITY AND ANGULARITY

The engineering behavior of granular materials is dependent upon the shape of the particle as well as size. For example, Peckover (Ref. 24) cites the problem of "running rail" with uncrushed gravel ballast as an example of the effects of particle shape on performance. Since a quantitative ranking of the geometric features of individual particles is both difficult and tedious, most engineers have identified the shape characteristics in general terms. Identification is usually taken for an overall view of all the particles which will fit into one of the following categories: rounded, subrounded or subangular, and angular. However, research efforts in the geologic sciences and concrete technology have made aids available that are capable of defining the particle shape characteristics more accurately. The most commonly used terms describing the geometric features of individual particles are sphericity and angularity or roundness. The sphericity is related to the proportion between length and breadth of the images, and the roundness by the curvature of the image edges (Ref. 35).

Three classes of grain shape are defined as bulky grains, flaky or scale-like grains, and needle-like grains. Bulky grains have the length, width, and thickness of the particles that are of the same order of magnitude. Flaky or elongated particles are defined as those whose least dimension is less than 60% of its mean size (Ref. 1). Each of these terms can be defined numerically by sphericity and roundness.

Sphericity is a term used to describe the difference between length (L), width (B), and thickness (H). Wadell (Ref. 36) defines the equivalent diameter of the particle (D_e) as the diameter of the sphere whose volume is equal to the particle volume (V), that is,

$$D_e = \sqrt[3]{\frac{6V}{\pi}} \quad . \tag{3-1}$$

Then, sphericity (X) is defined as follows:

$$X = D_e/L \quad . \tag{3-2}$$

A sphere has sphericity of unity, while flat or elongated particles have values less than unity. Flatness (F) is defined as :

$$F = B/H \quad . \tag{3-3}$$

The elongation (E) is defined by;

$$E = L/B \quad . \tag{3-4}$$

Zingg (Ref. 37) and Mather (Ref. 38) have proposed classification systems which numerically define the boundary values between spherical flat and elongated particles. Powers (Ref. 39) defines the sphericity factor to express the degree to which a given irregular particle departs from sphericity. The emphasis was placed in relating the sphericity factor to void ratio rather than quantitatively defining round or flat particles.

The methods of test for flat and elongated particles in aggregate are:

Corps of Engineers Designation, Coarse.................CRD-C119-53

Corps of Engineers Designation, Fine...................CRD-C120-55

A flat particle is defined as one having a ratio of width to thickness greater than 3. The length, width, and thickness, are, respectively, the greatest, intermediate, and least dimensions of any particle as measured along mutually perpendicular directions. They should be considered as the principal dimensions of the circumscribing rectangular prism.

A procedure for determining the amount of "flaky" or elongated particles has been developed and is covered by British Standard 812 (Ref. 40). A flaky particle is defined as one whose least dimension is less than 60 percent of its mean size.

The Canadian National Railways specifies that ballast "shall be free from an excess of thin or elongated pieces" and indicates that the amount shall be no greater than 30 percent (Ref. 24). A great variation (25 per cent of mean) was found among tests of in-service ballast (Ref. 24). Rad, (Ref. 41) found "flaky" particles tended to break down more rapidly in an impact test than did rounded particles.

The definition of <u>flaky</u> (thin or elongated) particles varies, but generally some ratio such as width to thickness, length to width, or at 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) (Ref. 24). 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 (Ref. 24).

Raymond, et al. (Ref. 42) numerically determined the Flakiness Index and Elongation Index by British Standard 812 (Ref. 40), and the sphericity and roundness by Pettijohn (Ref. 35) for ten CNR ballast materials, and then related these values to observations of lateral track stability made by CNR personnel. From this study he suggested that limits to be used as a stability guide were an Elongation Index of not greater than 10% and a sphericity of 0.55 to 0.70. However, the range of sphericity values was only 0.50 to 0.70 in the study. Specifications for flakiness index and roundness limits could not be determined. The validity of these limits still has to be established.

Angularity or roundness is a measure of the sharpness of the edges and corners of a particle and is independent of shape. Pettijohn (Ref. 35) defines roundness (R_o) for a two-dimensional projection of the particle as

$$R_o = \sum_{i=1}^{N} \frac{R_i/N}{R} \qquad (3\text{-}5)$$

in which R_i is the individual radius of a corner or edge, N is the number of corners or edges, and R is the radius of the maximum inscribed circle. When the corners and edges are sharp, their average radius is small and the roundness is low, but when the average radius of the corners approaches that of the inscribed circle, the roundness value approaches 1.0. By definition, a sphere has a roundness of 1.0 as well as a sphericity of 1.0. Other objects, however, which are nonspherical, may also have a roundness of 1.0, for example, as a capsule-shaped body, which is essentially a cylinder capped by two hemispheres (Ref. 35).

Visual estimates of particle roundness and sphericity are normally used in lieu of the more laborious method of image measurement. Charts to aid in this estimating are available in Ref. 43 and 44. In general, although estimates of individual grains may vary significantly, average values based on 50 or more particles tend to be similar, because the errors of estimation are largely compensating in the absence of strong operator bias (Ref. 35).

Although it is possible to separately measure 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 (Ref. 1). Huang (Refs. 45 & 46) 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. Huang (Ref. 45) compared the value of particle index with sphericity calculated according to the method developed by Wadell (Ref. 36) for gravel and crushed stone and found a good relationship between the two values for both materials. Recently Huang (Ref. 47) modified the particle index test to use a standard CBR mold, and the test has been adopted as a tentative ASTM standard.

Ishai and Tons (Ref. 48) 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.

Differences in the shape, angularity, and surface texture of aggregate may be due to geologic factors and production methods (Ref. 1). 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.

3.3 SPECIFIC GRAVITY AND ABSORPTION

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 (Ref. 1). ASTM (Ref. 26) defines bulk specific gravity as the ratio of 1) the weight in air of a given volume of impermeable portion of a permeable material (that is, the volume of solid matter including its impermeable pores or voids) at a stated temperature to 2) the weight in air of an equal volume of distilled water at a stated temperature. The expression generally used to compute apparent specific gravity (G_s) is

$$G_s = \gamma_s/\gamma_w \quad , \tag{3-6}$$

in which γ_s is the density of the solid particles, and γ_w is the density of water. Absorption is the ratio of a) the difference in weight of saturated surface-dry sample and oven dry sample in air to b) the weight of the oven dry sample in air. Average values of bulk specific gravity and absorption are available in Refs. 49 and 50. Values range from about 2.50 to 3.3 for bulk specific gravity, and from about 0.2 to 1.8 for 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. It should be noted that variations of more than 20 percent for the specific gravity of a particular type of aggregate are not uncommon.

The methods of test for specific gravity and absorption of coarse aggregates is:

AASHTO Designation. T85

ASTM Designation C127

Corps of Engineers Designation CRD-C107-60

Federal Specification Designation . . . . . Method 209.0

The field method of test for absorption by aggregates is:

Corps of Engineers Designation CRD-C109-48

This method of test is intended for use in making an approximate determination of absorption (of moisture) by fine or coarse aggregate.

Although specific gravity is not necessarily related to aggregate behavior, Wadell (Ref. 51) indicates that three individual rock types having low specific gravities (shale, sandstone, and chert) may display poor performance in concrete. Aughenbough, et al.(Ref. 44) had a similar conclusion based upon degradation of four different base course materials during compaction. That is, degradation increased with decreasing bulk specific gravity. Raymond, et al (Ref. 42) studies of breakdown field rating and bulk specific gravity of ballast materials showed less breakdown for materials having a higher specific gravity. However, good correlation for breakdown field rating and absorption could not be established. A minimum value of 2.65 was suggested.

Brink (Ref. 52) reported that for a specific rock source, the absorption of the rock will be greater and the specific gravity will be lower for the more weathered material. Since the absorption of water by an aggregate has a relationship with the pore volume of the aggregate, Bloem

(Ref. 53) believes it is more logical to evaluate the freeze-thaw resistance by using the simple absorption test rather than by the sulfate soundness test.

Canadian National Railway specifies the percentage absorption of ballast material should not be greater than 0.5%, and a minimum of 2.65 for the specific gravity of ballast (Ref. 24).

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 (Ref. 1). Peckover (Ref. 24) 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 (Ref. 54) found high testing variations (as much as 60 percent of the acceptable limit of 0.5 percent) for the absorption test. He also concluded that the absorption limit may allow approval of ballast which may break down too rapidly due to field weathering. Raymond, et al (Ref. 55) found good correlation between the freeze-thaw, soundness, and absorption test results.

3.4 REFERENCE DENSITY AND VOID RATIO

Multiparticle systems consist of three distinct phases: solid particles, and void space containing air and/or water. The compactness or packing is usually defined by the void ratio, the porosity or the density of the bulk sample. The void ratio (ratio of volume of voids to volume of solids) along with porosity (ratio of volume of voids to total volume) and density (ratio of total weight to total volume) are affected by the compactive effort imparted to the aggregate mass, as well as the aggregate shape, surface texture, and gradation. Density, in addition, is a function of the specific gravity of the minerals. In general, the void ratio at a given compactive effort is lower for well-graded than for uniformly graded materials, and for rounded grain than for angular grain material.

Powers (Ref. 39) investigated the void ratio in different combinations of aggregate and found that if size groups are similar, the size range governs the void content. He also found that small void ratios are possible if large single-size fractions are combined with small single-size fractions as well as if a well-graded material is considered. Lower void ratios are also an indicator of higher stability and lower compressibility characteristics of the material.

Specifications for determining a reference density and void ratio for concrete aggregates are:

Unit Weight of fine, coarse, or mixed aggregates:

AASHO Designation T19

ASTM Designation C29

Corps of Engineers Designation CRD-C106-57

Federal Specification Designation . . . . Method 201.0

Voids in Aggregate (for Concrete):

AASHO Designation T20

ASTM Designation C30

Corps of Engineers Designation CRD-C110-48

AREA recommends a minimum compacted unit weight for ballast of 70 to 100 lb/cu ft. (1120 to 1600 kg/m^3) for slag ballast materials (Ref. 2).

Heavy ballast is generally preferred to lightweight ballast because it is thought to offer more resistance to lateral movement (Ref. 24).

Little meaningful data has been obtained on the in-place void ratio, porosity or density of ballast because suitable methods of measurement have not been available. Part of the effort in this present research has been to develop appropriate methods and to use them to study the density state of ballast in the field. The methods, including a reference density test, are described in Ref. 23.

Relative density* expresses the degree of compactness of a material with respect to the loosest (minimum density) and the densest (maximum density) conditions of the material that can be attained by specified laboratory procedures. Thus a material in the loosest state would have a relative density of zero and in the densest state, of 100 percent. The magnitude of the dry unit weight of a cohesionless material does not, by itself, reveal whether it is loose or dense, because of the influence of particle shape and gradation on this property. ASTM D 2049 (Ref. 26) is a standard specification used to determine the maximum and minimum void ratios for calculating relative density. The types of errors in determining relative density and a comprehensive review of these methods is given in Ref. 56. However, this method is not suitable for ballast materials.

3.5 HARDNESS AND TOUGHNESS

Moh's scale ranging from 1 for talc to 10 for diamonds, provides a relative measure of mineral hardness. It was developed in 1812 and is now being used successfully for correlating physical and technological properties of rocks. Approximately 70 percent of the crushed rock produced in the United States is in the range of 3 to 4 on this scale (Ref. 57).

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* Actually more correctly termed relative void ratio because it equals the ratio of a) the difference between the maximum and sample void ratios divided by b) the difference between the maximum and minimum void ratios.

The hardness of rocks, on a broad range, can be covered by the following four categories (Ref. 29):

Soft	Medium	Hard	Very hard
Asbestos rock	Limestone	Granite	Taconite
Gypsum rock	Dolomite	Quartzite	Granite
Slate	Sandstone	Iron ore	Felsite
Talc		Traprock	Traprock
Limestone			

The toughness test is made to determine the resistance of a rock to impact. Available standard test methods are:

Corps of Engineers Designation CRD-C132-53

Federal Specification Designation . . . . . Method 207.0

Average values of toughness of the principal kinds of rock are given in Ref. 49. The range is about 6 to 20.

Several other testing schemes have been developed for use in estimating the hardness of individual aggregate particles. The British use British Standard 812 in which the aggregate is confined in a mold and subjected to 3170 psi (21.86 MN/m^2) pressure (Ref. 58). The amount of breakdown or change in gradation is used to calculate the "crushing value." Scalzo (Ref. 58) 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.

Raymond, et al. (Ref. 42) determined the "crushing value" for 10 ballast materials using B. S. 812. A good correlation between breakdown field rating and "crushing value" did not exist. However he suggested a limit of not greater than 25%.

Rad (Ref. 41) 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. He found rounded particles to be less susceptible to impact than were sharp particles, which were in turn less susceptible than flat or elongated particles.

3.6 CHEMICAL SOUNDNESS

The chemical soundness test for aggregates is used to determine their resistance to disintegration by subjecting the aggregates repeatedly to saturated solutions of sodium sulfate or magnesium sulfate. This method furnishes information helpful in judging the soundness of aggregates subject to weathering action, particularly when adequate information is not available from service records of the material exposed to actual weathering conditions. Test results by the use of the two salts differ considerably, so that care must be exercised in fixing proper limits in any specifications which require these tests.

Test specifications for soundness of aggregate by use of sodium sulfate or magnesium sulfate are:

AASHO Designation T104

ASTM Designation C88

Corps of Engineers Designation . . . . CRD-C137-62 and C115-55

Federal Specification Designation . . . . . Method 203.01

Soundness tests such as ASTM C88 and AASHTO T104 consist of five cycles of alternately soaking the aggregate in the saturated solution followed by drying. Soundness is evaluated by calculating the loss or change in gradation as a percentage of the initial weight. Factors affecting the results of the soundness test include type of solution, number of cycles, void characteristics of particles, mineralogy of aggregate, and presence of fractured surfaces (Ref. 1).

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 (Ref. 1). Additionally the soundness test has been criticized because it is not reproducible in different laboratories, especially if sodium sulfate is used. West, et al.(Ref. 59) concluded the sodium sulfate test did not prove useful or reliable. Bloem (Ref. 53) 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 (to be discussed in a following section) apparently give results which predict the weathering resistance of aggregate better than does the soundness test (Refs. 24, 59, and 54). However, Raymond, et al.(Ref. 55) found good correlation between field performance, the sodium sulfate soundness and the freeze-thaw (AASHTO T103) tests. He suggests a value of sulfate soundness not greater than 1.25% (Ref. 42). Ballast specifications for sodium sulfate soundness reported by others are:

	Source	Soundness Not to Exceed
1.	AREA	10%
2.	Canadian National Railways (CNR)	5%
3.	Canadian Pacific Railways (CPR)	10%
4.	Southern Pacific Transportation Co. (SPTC)	10%

3.7 ABRASION RESISTANCE

Two testing procedures have been devised to measure the resistance of rock to abrasive wear. The Los Angeles abrasion test first came into use in 1932, and it is now a requirement in most of the states. The Deval test was developed by the French in 1878 and subsequently modified in 1937. It has been found to be of less value than the Los Angeles test as an indication of the behavior of aggregate in actual use (Ref. 29).

The specification for the Los Angeles procedure for the testing of crushed gravel, rock, and slag or uncrushed gravel for resistance to abrasion is:

AASHO Designation T96

ASTM Designation C131

Corps of Engineers Designation CRD-C117-46

Federal Specification Designation . . . . . Method 208.11

Ledge rock, when hand broken into approximately cubical fragments of the specified sizes and tested in this manner, has been found to have a loss of approximately 85 percent of that for crushed rock of the same quality.

The Los Angeles abrasion testing machine consists of a steel cylinder in which the material is rotated, together with cast iron or steel spheres, for 500 to 1,000 revolutions. The sample may be either wet or dry. British Railways uses the wet mode as a standard for ballast approval. The abrasive wear is reported as the difference between the retained weight on the designated screen and the weight of the original sample, expressed as a percentage of the original weight.

Average wear values for principal types of rock range from 14 to 47 (Ref. 49). Among the factors influencing the Los Angeles test results are number of revolutions, sample moisture conditions during test, uniformity of aggregate, aggregate shape, gradation, and mineralogy (Ref. 1).

AREA specifications recommend that LA abrasion losses be limited to a maximum of 40 percent (Ref. 2). 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 (Ref. 1). West, et al. (Ref. 59) obtained good results by running both wet and dry tests. Scalzo (Ref. 58) found that the values correlated well except for basalt. According to Goldbeck (Ref. 60) the Los Angeles abrasion test "gives a pretty fair indication of the service value of ballast." Dalton (Ref. 54) found that the Los Angeles abrasion test alone was not an entirely reliable indicator of ballast quality. He also indicated that ballast with abrasion losses slightly in excess of the maximum allowable may be satisfactory if the resistance to freeze-thaw is good.

Raymond, et al., (Ref. 42) reports that a good correlation between breakdown field rating and Los Angeles abrasion does not exist for the 10 ballast types studied. Limiting values tabulated by Raymond et al. (Ref. 55) for other railways are shown below:

Source	L. A. Abrasion Limit
AREA	40%
CNR	20%
CPR	40%
SPTC	25%

The specification for the Deval procedure for the testing of crushed gravel, rock, and slag or uncrushed gravel for resistance to abrasion is:

AASHO Designation . . . . T3 and T4-35

ASTM Designation . . . . D2-33 (1968) and D289-63

Corps of Engineers Designation CRD-C141-56

Federal Specification Designation . . . Method 208.0

The Deval abrasion testing machine consists of a hollow iron cyclinder in which the material to be tested is rotated 10,000 revolutions with an abrasive charge of six cast-iron spheres. The difference between the retained weight on the designated screen and the weight of the original sample, expressed as a percentage of the original weight is the loss by abrasion. The test is still specified, but the method is essentially obsolete. Average wear values for principal types of rock are 2.5 to 10.0 according to Ref. 29.

3.8 FREEZE-THAW CHARACTERISTICS

The procedure to be followed in testing aggregates to determine their resistance to disintegration by freezing and thawing is:

AASHTO Designation T103

This test furnishes information helpful in judging the soundness of aggregates subjected to weathering action. However, in selecting aggregates, principal dependence should be placed on service records of the materials when exposed to actual weathering conditions. In the absence of such information, the test method affords a useful guide. Because of the limited amount of information concerning the significance of the test results and because of the lack of information as to the uniformity of tests made in different laboratories, the test method should not be used arbitrarily for rejection; it should be used only to furnish information to indicate whether or not the materials require further investigation of their soundness (Ref. 29).

Several versions of freeze-thaw tests have been used to evaluate aggregate resistance to disintegration by cycles of freeze-thaw (Ref. 1). In general, either partial or complete immersion can be used, and the fluid can be either water or a water-alcohol solution. Although the AASHTO procedure does not recommend the number of cycles, some authorities have recommended the use of as many as 50 freeze-thaw cycles. Raymond, et al. (Ref. 42) has used 58 cycles with a 48 hour period per cycle. Resistance is measured by the change in gradation caused by the 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. However a high number of cycles or a long cycle time is impractical and uneconomical in most approval tests because of the time and cost involved.

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 (Ref. 24) and West, et al. (Ref. 59) feel the freeze-thaw test yields results useful for predicting resistance to field weathering (Ref. 1). Raymond, et al., (Ref. 42) did not find a good correlation between breakdown field rating and freeze-thaw values for 10 ballast materials, but suggests 3.0% as a limiting value.

3.9 OTHER PROPERTIES

Several other tests with some relevance to ballast are briefly summarized below. All are of interest in aggregate selection and application.

1. Production of Plastic Fines in Aggregates

The durability factor is a value indicating the relative resistance of an aggregate to producing detrimental clay-like fines when subjected to the prescribed mechanical methods of degradation. In the California Highway Department test (California Test No. 229) the durability is measured by agitating aggregate in water and measuring the change in gradation.

The method for determining the durability factor of aggregates is:

AASHTO Designation T210

California Designation 229-C

2. Plasticity of Fines

Excessive fines content normally is not permitted for high quality ballast materials. However, 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 T 90) can be used to determine the plasticity characteristics of the faction finer than the number 40 sieve (0.425mm).

Typical ballast specifications do not contain any limitations concerning plasticity characteristics (Ref. 1). Subballast materials in general follow limits set by ASTM D 1241 for soil-aggregate mixtures, which requires that the liquid limit be less than 25 and the plasticity index be less than 6.

3. Percentage of Particles of Less than 1.95 Specific Gravity in Coarse Aggregates

The method of test for the approximate determination of shale in coarse aggregate is:

AASHTO Designation T150

The test separates, along with the shale, other particles of low specific gravity or relatively high surface area such as iron oxides, soft particles, and other small materials.

4. Clay Lumps in Aggregates

The method of test for the approximate determination of clay lumps in the routine examination of aggregates is:

AASHO Designation T112

ASTM Designation C142

Corps of Engineers Designation CRD-C142-65

Federal Specification Designation . . . . . . Method 205.0

Clay lumps and friable particles are limited to 0.5 percent by the AREA specification (Ref. 2). ASTM C142 describes clay lumps and friable particles as particles than can be broken with the fingers after the aggregate has been soaked in water for 24 $\pm$ 4 hours (Ref. 26).

5. Soft Particles in Coarse Aggregates

The method of test for determining the quantity of soft particles in coarse aggregates on the basis of scratch hardness is:

AASHO Designation T189

ASTM Designation C235

Corps of Engineers Designation CRD-C130-57

Federal Specification Designation . . . . . . Method 228.0

This test is to be used to identify materials which are soft, including those formed of a soft material and those which are so poorly bonded that the separate particles in the piece are easily detached from the mass. The test is not intended to identify other types of deleterious materials in aggregate.

A study of tests for the determination of soft pieces in aggregate gave as a conclusion, that the scratch-hardness test using a hard yellow brass scribe was the only one considered suitable for laboratory and field use (Ref. 61). 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 (Ref. 2).

6. Method of Test for Clay Lumps and Friable Particles in Aggregates (Slaking Test Method)

The method of test for determination of clay lumps and friable particles in aggregates by slaking in water is:

Corps of Engineers Designation CRD-C118-55

7. Petrographic Analysis

Petrographic analysis (ASTM C295) is the microscopic examination of rock samples and the identification of the constituents. The CNR commonly uses this method of analysis for ballast characterization (Ref. 24).

Several investigators (Refs. 24, 59, 62, 63) have attempted to correlate the results of petrographic analysis of the particles with field degradation. 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 (Ref. 1).

The method for the petrographic examination of representative samples of materials proposed for use as aggregates in concrete is:

ASTM Designation C295

Corps of Engineers Designation CRD-C127-64

The specific procedures employed in the petrographic examination of any sample will largely depend on the purpose of the examination and the nature of the sample (Ref. 29).

Nomenclature has been prepared to provide brief, useful, and accurate descriptions of some of the more common or more important natural materials found as constituents of mineral aggregates. These are specified in:

ASTM Designation C294

Corps of Engineers Designation CRD-C139-56

The descriptions provide a basis for common understanding when they are used to designate aggregate constituents. These descriptions are not adequate to permit the accurate identification of the natural constituents of mineral aggregates. In many cases, this identification can only be made by a qualified geologist, mineralogist, or petrographer using the apparatus and procedures of these sciences (Ref. 29).

Identification of the constituent materials in a mineral aggregate may facilitate the recognition of its properties, but identification alone, however accurately it may be accomplished, is not a sufficient basis for predicting the behavior of aggregates in service. Mineral aggregates composed of constituents of any type or combination of types may perform well or poorly in service depending upon the exposure to which they are subjected, the physical and chemical properties of the matrix in which they may be embedded, their physical condition at the time they are used, and upon other factors (Ref. 29).

3.10 BALLAST IDENTIFICATION

Ballast can be categorized as crushed stone or rock, processed gravel, pit-run gravel, and slag. Crushed stone, as an aggregate produced from processed rock, is identified with the associated geologic rock source.

A great deal can be gained from a knowledge of rock identification. In addition to their varying intrinsic properties, rocks may be affected in different ways by the aggregate processing itself. Some rock types, after processing, will have better finished shapes than others. Granite, for example, has a good quality for aggregate and yields a good finished product, although it is the source material for less than 8 percent of quarried aggregate. Approximately 11 percent of all quarried rock is basalt, diabase, and gabbro, a group generally known as "traprock." These are dark, heavy rocks with a tendency to crush to flat pieces in the finer sizes. Limestone and dolomite are the most widely found common quarry rocks and account for approximately 71 percent of the total. These high-calcium stones usually crush to yield well-shaped aggregate and lend themselves well to processing by impact breakers and hammer mills. Sandstone may vary greatly in its physical properties and must be checked carefully before it is used as an aggregate. Approximately 4 percent of quarried aggregate is produced from this type of rock (Ref. 64).

It is becoming increasingly important to include detailed petrographic examination in the evaluation of any raw material for use as ballast. Such an analysis as a quality test by qualified personnel is both quick and reliable. In order to make a petrographic examination of rock samples for aggregate, it is necessary to be able to identify the various rock types. Information for identification may be found in Ref. 65.

Processed gravel is pit-run gravel which is washed and screened and/or crushed. These materials are found as gravel deposits formed as a result of glacial or fluvial action. Identification is usually restricted to particle size and shape characteristics rather than identifying the mineral constituents or rock type. ASTM D2488 (Ref. 26) is a standard method used to visually identify and describe such materials.

Slags used as ballast and subballast aggregates are principally the nonmetallic by-products of metal-making furnaces. Identification is associated with the type of processing, i.e., air- or water-cooled, and type of metal ore processed, such as copper or nickel. Three general types are air-cooled blast-furnace slag, granulated blast-furnace slag, and steel or open-hearth slag.

4. MECHANICS OF GRANULAR MATERIALS

The design and performance of track structures is influenced by the static and dynamic stress-deformation and strength characteristics of the ballast and the underlying soil layers. The mechanical properties representing the material response is determined either from tests on specimens in the laboratory under the anticipated field stress conditions, or from tests conducted in the field on undisturbed material samples. In this chapter the parameters defining the stress-deformation and strength characteristics will be described, the methods by which they are measured will be explained, and the factors influencing the properties will be discussed.

Soil is a material whose behavior is non-linear and inelastic, as well as stress-history and strain-rate dependent. Hence, the stress-deformation and strength behavior is complex, so that for practical applications, a simplified representation in the choice of the mechanical properties is always necessary.

Using the soil classification systems common to geotechnical engineering, ballast materials will generally be categorized as having their primary component in the gravel-size particle range. AREA specifications (Ref. 2) for crushed stone, crushed slag, and crushed gravel ballast indicate uniformly-graded material, whereas those specifications for pit run gravel are for a well-graded material. Pit run gravel contains sand-size as well as gravel-size particles. Furthermore, degradation of particles or infiltration of fine material can convert a uniform, gravel-size ballast to a material which includes sand and silt-size particles. Thus, although particles larger than sand size are normally desired in ballast, ballast materials can encompass most of the gradation and particle sizes of granular soils.

An abundant amount of information is available in the literature on the behavior of granular soils, although much more relates to sand-size materials than to gravel-size materials. These materials are identified primarily by size classification, with minor emphasis placed on the geologic origin and compositional characteristics of the material. Unfortunately, the latter factors are quite important for ballast materials. Because only a small percent of available information is directly derived from tests on ballast materials, this chapter will present a general treatment of the mechanical behavior of non-cohesive or granular materials, that is, all of those primarily composed of sand and gravel.

4.1 STRESS-DEFORMATION AND STRENGTH BEHAVIOR

Basic Concepts. Granular soils may be visualized as an assembly of particles which can be combined in a variety of packing arrays with a range of density or void ratio. The voids between the particles are filled either with air or water or both. When a granular material is stressed, it will deform by a combination of: 1) compression of individual particles and breakdown at the particle contact points, and 2) movement of particles relative to each other. This movement is resisted by friction at the contact points, plus interference between particles through interlocking. Both mechanisms of resistance are a function of the stress level, with the consequence that the resistance to deformation under stress increases with the mean stress level, all other factors being equal. Stresses applied to a granular soil are transmitted through either the contact points or the pore fluid. The average stress on a cross-section through the soil has been shown to be appropriately represented by the sum of the effective or intergranular stress and the pore pressure.

Because the pore pressure has no shearing resistance, the effective stress controls the strength and mechanical behavior of the soil.

In a partially saturated granular soil, the equivalent pore pressure is essentially equal to the pore air pressure, which is usually atmospheric and hence taken as zero. In such a case the effective stress equals the total stress. For a soil that is essentially saturated, that is, with all of the voids full of water, the equivalent pore pressure equals the pore water pressure. Because the soil particles and the water are both incompressible compared to the soil skeleton, when a saturated sample is stressed, the pore water pressure can change significantly, and hence alter the magnitude of strength and stiffness. Such changes will not occur if adequate drainage is allowed so that the pore water pressure will remain constant, usually at atmospheric pressure, and hence taken as zero. Thus, again, the effective stress equals the total stress.

During loading, a soil may be drained, undrained, or partially drained with respect to movement of pore fluid. In the undrained case the boundary of the soil is restricted so that no pore fluid can flow in or out of the sample. In the drained case pore fluid can move in or out of the sample to accommodate volume changes of the soil structure with no pore pressure change. The partially drained case is, of course, the situation intermediate to the other two.

A partially saturated or dry granular soil will behave the same whether it is drained or undrained during loading, because the compressibility of the pore air is sufficiently great to result in little change in pore pressure when the soil structure changes in volume. In the drained condition, saturated soils will also behave about the same as those that are partially saturated or dry, because the flow of pore water accommodates the volume change without any pore pressure change. However, the mechanical properties will be very different for saturated, granular soils in undrained loading, because of the change in pore pressure which will result in a change in effective stress.

In the case of a typical ballast material without fines and with the underlying boundaries open to drainage, the ballast will have a low degree of saturation, and the total stresses will be equal to the effective stresses. With the lateral and underlying boundary sealed, the ballast can become saturated, but if the loading is slow enough to permit upward drainage of pore water, the ballast will still behave as a drained material. However, when the ballast becomes clogged with fine materials, the permeability may become small enough that the rate of dissipation of pore pressure is slow enough during dynamic loading that the ballast will behave in an undrained manner with a buildup of pore pressure and, consequently a significant alteration in the mechanical behavior.

The most common method of measuring the stress-strain and strength properties of a soil is the triaxial test illustrated conceptually in Fig. 4.1. A cylindrical sample of soil is surrounded by an impermeable membrane and placed in a fluid-filled chamber, so that an all-around confining pressure (σ_3) can be applied to the outside of the sample. An additional axial compressive stress ($\sigma_1 - \sigma_3$) is then applied to the sample, causing a compressive deformation under constant total lateral stress. The resulting stress state is represented by an axial total stress (σ_1) and a lateral total confining stress (σ_3).

Typical stress-strain curves from such a test are illustrated in Fig. 4.2. The dense soil shows a significantly higher strength than the loose soil at

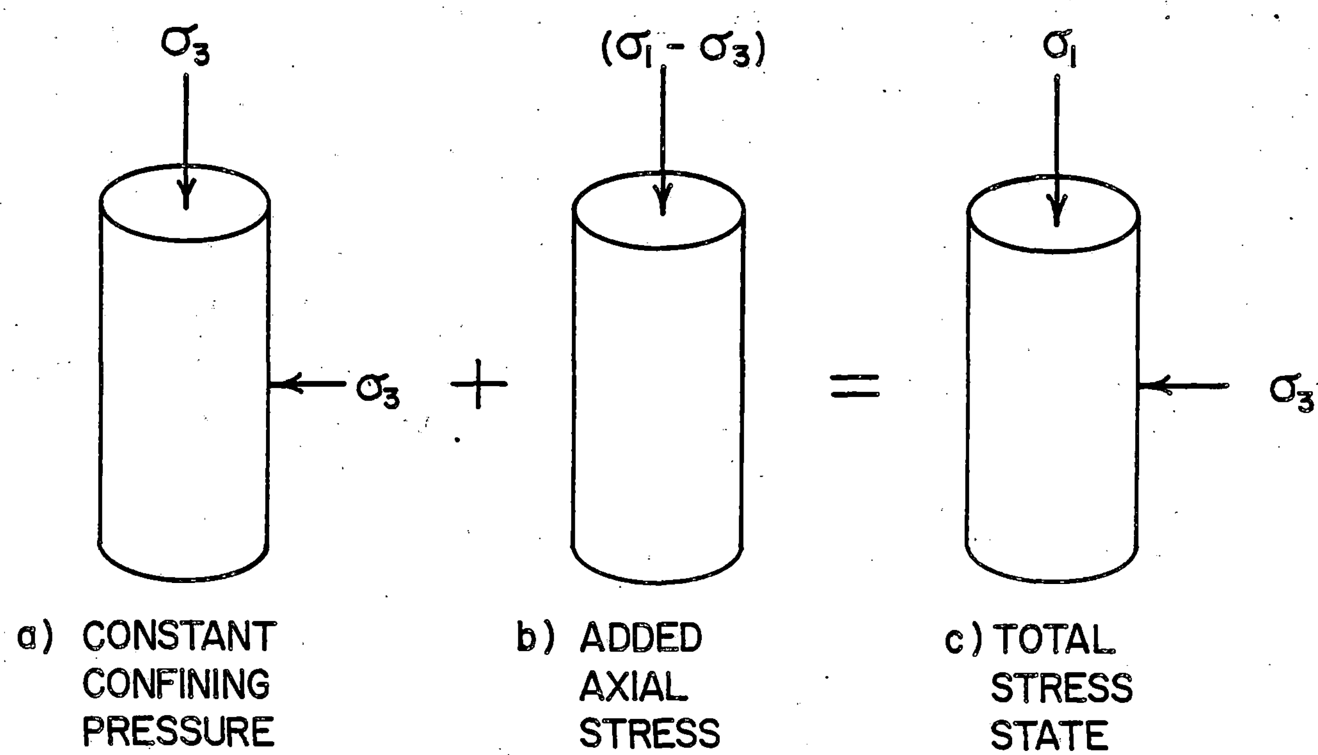


Figure 4.1. Stress Conditions for Stress-Strain Determination of Soil in Triaxial Test

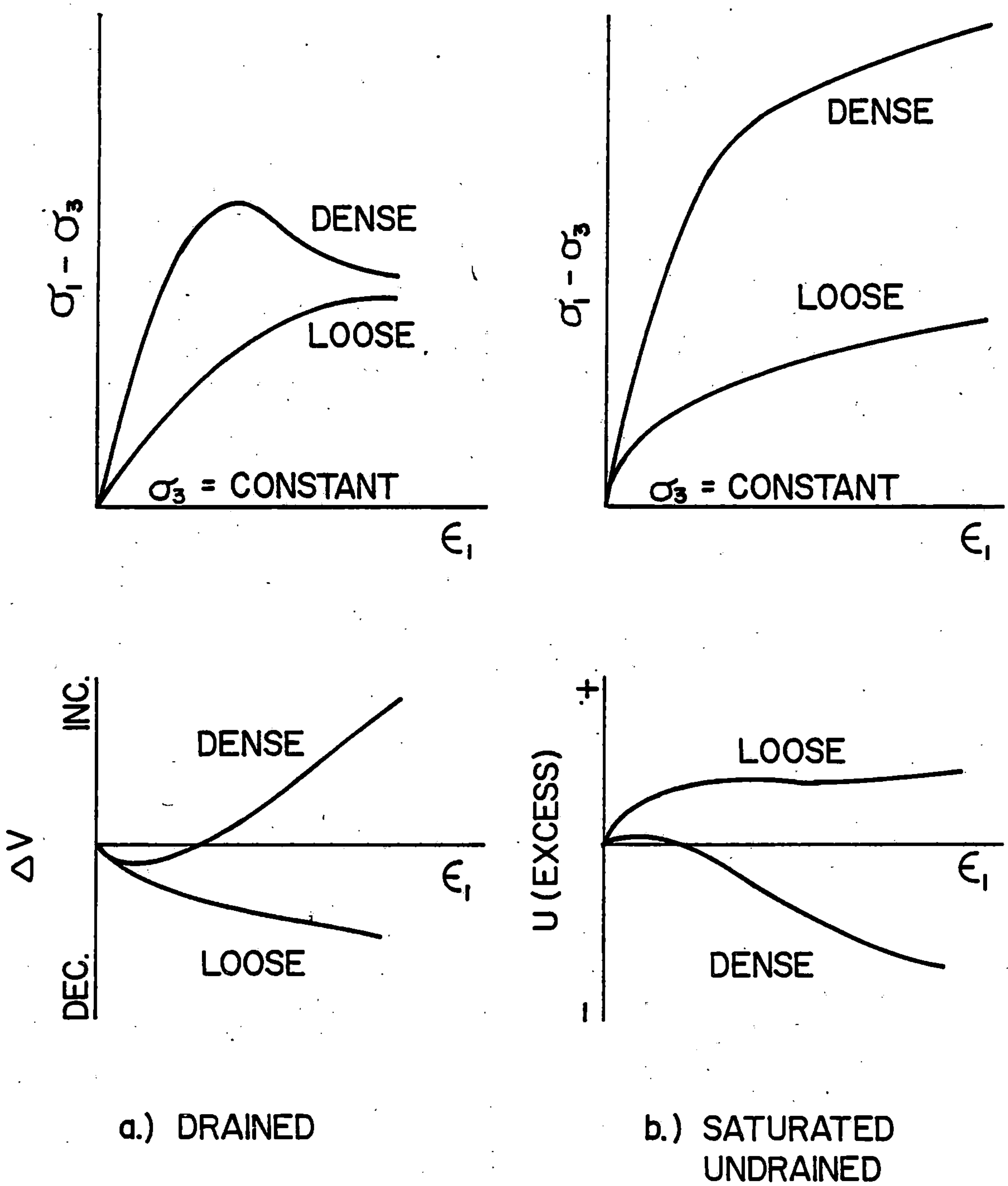


Figure 4.2. Typical Static Axial Stress-Strain Curves for Cohesionless Soils at Constant Total Confining Pressure

constant confining pressure. In the drained case, a volume change will occur during loading, with the dense sample at first decreasing in volume and then dilating to a volume greater than its initial volume; but the loose soil shows a continually decreasing volume. In contrast, for the saturated, undrained case, no volume change can occur. Instead, for the loose soil, the pore water pressure increases during loading, while for the dense soil, the pore pressure at first increases and then shows a significant reduction. This pore pressure change will alter the effective stresses so that the dense soil is stronger than in the drained case, while the loose soil is weaker than in the drained case.

If the drained tests are run at different total confining pressures, the stiffness and strength will increase as the confining pressure increases. The stress state at the peak points can be represented by Mohr's circles as shown in Fig. 4.3. For ideal behavior of cohesionless soils, the circles for different confining pressures will all be tangent to the same straight line through the zero stress state. Cohesion is assumed to be zero, and therefore the strength parameter is the angle of shearing resistance ϕ. In the undrained case where the pore pressure changes (Fig. 4.2b), the peak value of $\sigma_1 - \sigma_3$ will be different for the same value of σ_3. The drained and undrained Mohr's circles for the loose soil in Fig. 4.2 are compared in Fig. 4.4. A lower value of strength parameter is obtained for the undrained test when the results are represented in terms of total stress. However, the effective stress circle for the undrained case has the same strength parameter as that for the drained case. This is a consequence of the fact that strength is governed by effective stress rather than by total stress. The results for the dense soil would be analogous, except that in the dense case the

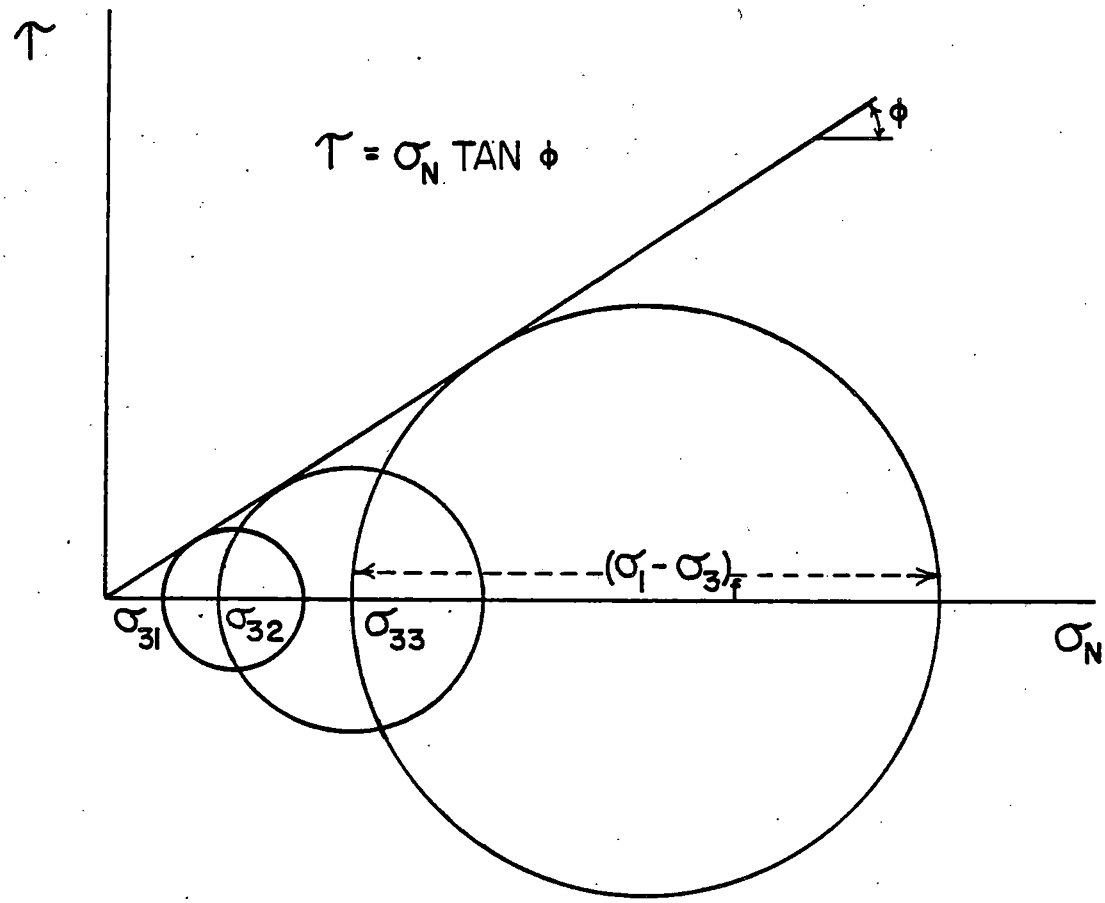


Figure 4.3. Strength Parameter Determination from Mohr's Circles for Drained Stress-Strain Curves

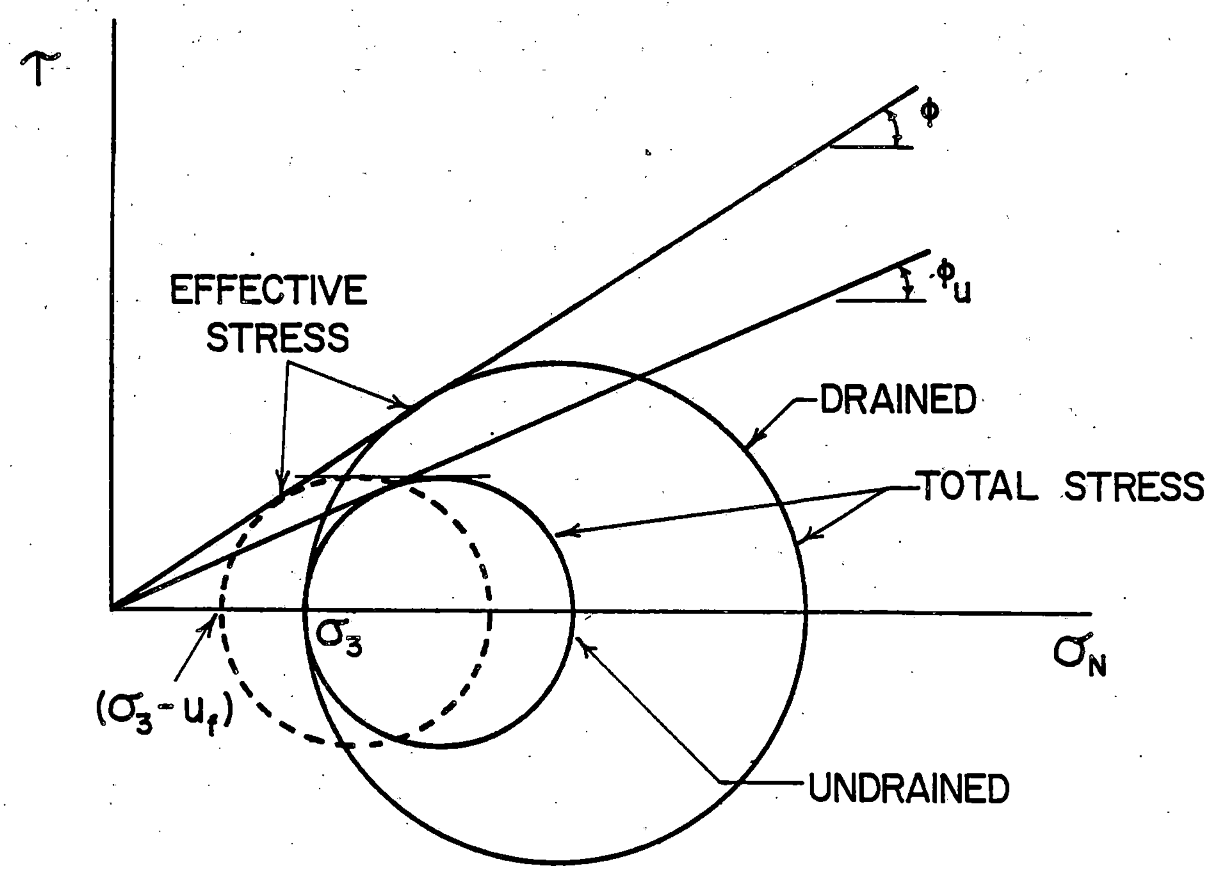


Figure 4.4. Illustration of Mohr's Circles for Drained and Undrained Strength for Loose Cohesionless Soil

undrained circle will be larger than the drained circle, and the effective stress circle for the undrained case will shift to the right of the total stress circle until it becomes tangent to the ϕ line.

Factors Influencing Behavior. Granular soil behavior in the laboratory and in the field is affected by characteristics of the soil type, imposed stress conditions, and type of drainage. Variability in test results on identically prepared specimens is accounted for by the preceding factors as well as by the limitations in the test procedures and equipment. The latter, however, will not be considered in this chapter. The main factors influencing cohesionless soil behavior are: 1) degree of compactness, 2) degree of confinement, 3) plasticity of the fine fraction, 4) degree of saturation or drainage conditions, 5) particle shape, 6) gradation curve characteristics, 7) particle size or principal component, 8) type of test and 9) rate and type of loading. The treatment of these factors in this chapter will be general and hold for a majority of cases, although specific conditions can be encountered which will contradict the generalized behavior.

Soil particles may be arranged in either a loose (high void ratio or low relative density) or dense (low void ratio or high relative density) state depending upon the amount of energy imparted to the soil. Dense soils have a higher modulus of deformation, a higher peak stress (compressive strength) and a lower strain at failure than loose soils (Fig. 4.2). After the peak stress has been reached, dense soils show a reduction in stress with increasing strain while loose soils often continue to show a constant stress or increasing stress with increasing strain. Upon loading, loose drained or unsaturated samples undergo a volume reduction and subsequent decrease in void ratio, whereas dense samples dilate and increase in void ratio. At some strain after failure both curves when extrapolated will asymtotically approach the same stress and will essentially have the same void ratio, which is defined as the critical void ratio.

Two samples identically prepared but tested under different confining pressures will show appreciable differences in behavior. While the failure strains are essentially the same, samples at higher confining pressures have a higher modulus of deformation and peak stress. The higher strength is primarily due to the additional interlocking of the grains under the higher confining pressure.

The presence of plastic fines tends to lubricate the larger particles such that a reduction of interparticle friction occurs. The result is a lower modulus of deformation, lower peak stress and higher failure strain than those existing for non-plastic fines (Fig. 4.5a). The differences in the two curves can be accounted for by the quantity and the degree of plasticity of the plastic material.

Since granular materials without fines are highly permeable, pore water cannot completely fill (saturate) the void space unless the flow is restrained. The behavior of soil that is both saturated and undrained during loading is quite different from that which is fully drained or unsaturated (Fig. 4.2). However, on an effective stress basis the results are essentially the same.

Angular particles interlock more than rounded particles and thus have a higher peak stress. As shown in Fig. 4.5b, the modulus of deformation is lower and the failure strain is higher for angular particles than rounded particles with uniform grading at the same relative density, but different void ratios. However, the influence of shape on failure

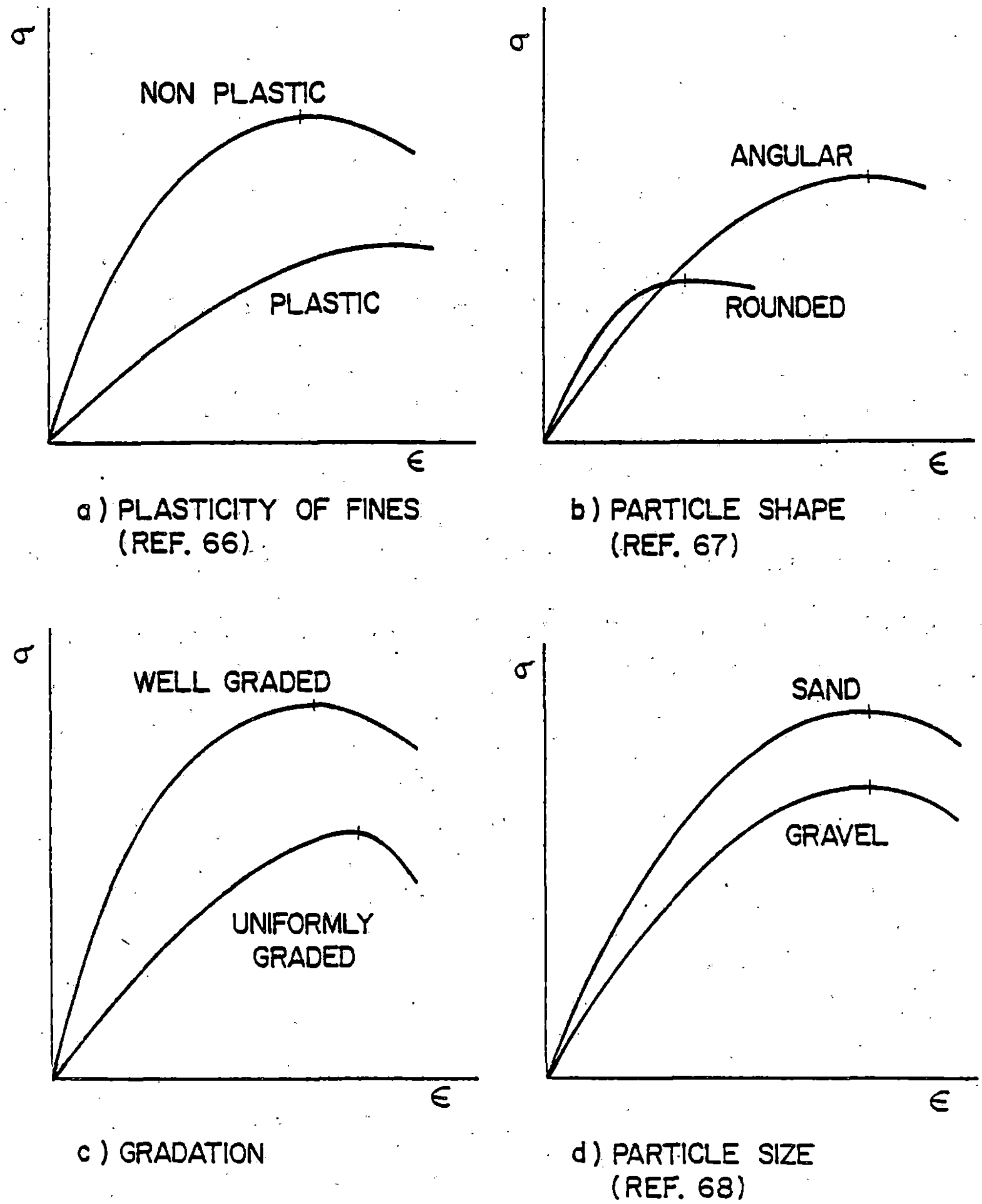


Figure 4.5. Illustration of the Effect on the Axial Stress-Strain Relationship of Plasticity of Fines, Particle Shape, Gradation, and Particle Size

strain and modulus values is not clearly established in the literature. The difficulty arises in obtaining samples of nearly the same grading, maximum and minimum void ratios, but different particle shape.

Well-graded and uniformly-graded materials represent the extreme grading limits. Well-graded soils have a larger number of contact points, thus producing a higher degree of interlocking and shearing resistance. These materials have a higher peak stress (Fig. 4.5c), but the comparison of modulus of deformation and failure strain is not well established, due to differences in the magnitude of the limiting void ratios and relative densities.

The angle of shearing resistance is highly influenced by the particle size. Reported trends are for both increasing and decreasing friction angle with decreasing particle size. Smaller size particles have more particle contact area, thus producing a more uniform distribution of shearing stresses within the soil specimens. Large particles have a higher degree of crushing and fracturing, but these effects also produce greater interlocking. An important factor to consider for comparison of stress-strain curves is denseness of particle packing. As shown in Fig. 4.5d, sand exhibits a higher peak stress than gravel, but this is for samples prepared at the same void ratio. The sand is in a dense state, whereas the gravel is in a loose state. This factor alone strongly influences the results and would be misleading for the comparisons made here. The proper parameter used should be relative density, which would give gravels the higher peak stress and modulus of deformation. Failure strain is essentially the same for the two materials.

The behavior of granular soils is a function of the method by which stresses are applied to the specimen. The isotropic or hydrostatic compression test, the confined or one-dimensional compression (oedometer) test and the trixial test each attempt to simulate the in-situ stress conditions. The results obtained from each are vastly different (Fig. 4.6a). In isotropic compression, the sample is subjected to the same stress on all sides with small strains accompanying increasing stress. Failure of the specimen does not occur. In the confined compression test, no lateral yield occurs during axial load, but strains are greater than in the isotropic compression test at the same stress level. In triaxial compression, lateral yield occurs during the application of the axial load under constant lateral stress. Strains are larger than in the two previous cases, and sample failure can occur.

Strain rate can have a significant effect on the magnitude of the response of granular soil to loading. Typical static strain rates are 1/120 to 1/30%/sec whereas dynamic strain rates are greater than 1/30%/sec and can exceed 1000%/sec. Specimens at high strain rates have higher peak stress and modulus of deformation and lower failure strain than samples at low strain rates (Fig. 4.6b). However, for granular soils the strain rate effects are not usually large, and can often be neglected.

Soils may be subjected to both static and dynamic loads. The latter can be either repeated, or cyclic, or high strain rate loads. Cyclic load testing usually involves the application of an alternating axial stress added to a hydrostatic state of stress, which causes shear stress reversal within the specimen (Fig. 4.6c). Repeated load testing is similar to cyclic loading except that the axial stress is usually alternated from zero to a maximum state of stress with no shear stress reversal occurring (Fig. 4.6d). The drained cyclic and repeated load tests show an increasing modulus with number of cycles, and dynamic modulus is usually greater than the static modulus.

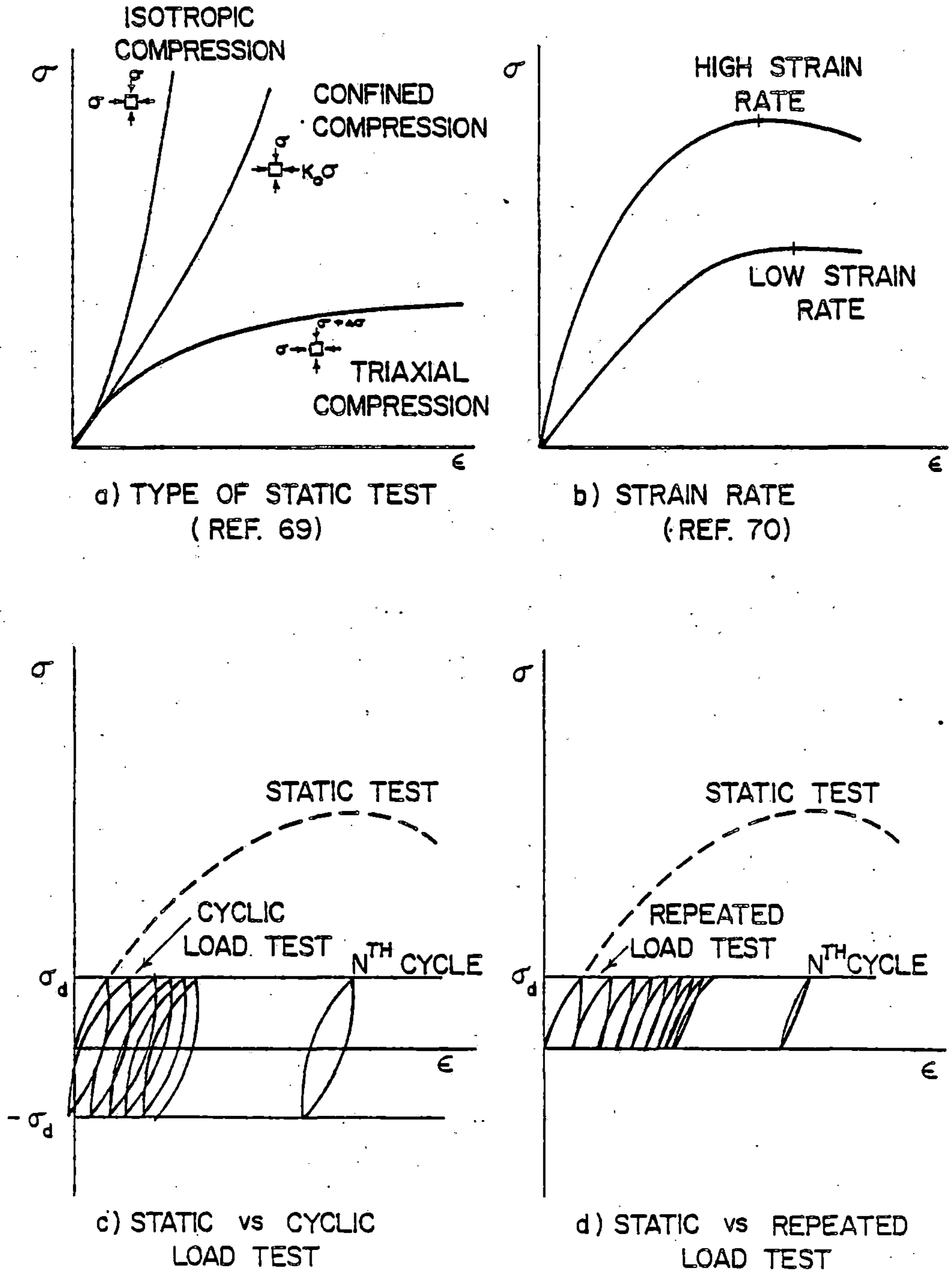


Figure 4.6. Illustration of the Effect on the Stress-Strain Relationship of Type of Test

A summary of the variables and the desired properties associated with stress-strain curves, such as peak stress, initial modulus of deformation and failure strain, are shown in Table 4.1. These variables are also ranked as to the importance of the parameter with respect to the effect on strength determination. As can be seen, the degree of compaction, confinement, saturation, and rate of loading affect the behavior of a given soil, while particle shape, plasticity of the fine fraction, gradation and soil type cause differences in the behavior of different soils. The variables influencing the behavior the most are level of compaction, degree of confinement, and soil type.

Theoretical Relationships. Several theories have been advanced to describe the stress-strain behavior of granular soils. The limiting factor is that verification of these theories is mainly for sandy type soils and not for gravel. Because gravels or ballast materials are as much a particulate system as sands, the theories based on sand behavior can be used as a reasonable approximation of the coarse-grained material behavior as well. Treatment of the static and dynamic behavior theories will be descriptive rather than a detailed presentation of equations involved.

Rowe (Ref. 71), and Newland and Allely (Ref. 72) both proposed static stress-dilatancy relations to define granular soil behavior. King and Dicken (Ref. 73) showed the two theories are founded on the same basic stress and volume change equations and only differ with the final assumptions made to allow interpretation of experimental results. The values of empirical effective interparticle friction angle, were essentially the same on dense sand samples in drained triaxial compression tests. Rowe's (Ref. 71) theory is accurate at high stress levels where slip strains

Table 4.1. Summary of Effects of Individual Variables on Stress-Strain Properties

Variable	Peak Stress	Initial Modulus of Deformation	Failure Strain		*
	Indicated Factor has Greatest Effect for:				
1. Degree of Compactness (Relative Density, Void Ratio) (a) Loose (b) Dense	(b)	(b)	(a)		V
2. Degree of Confinement (a) Low (b) High	(b)	(b)	same		V
3. Plasticity of Fine Fraction (a) 0 (b) PI = const	(a)	(a)	(b)		L
4. Degree of Saturation; Type of Drainage (a) S_o = 0% (b) S_o = 100%	(b)	(b)	(b)	total stress	L
	same	same	same	effective stress	U
5. Particle Shape (a) Rounded (b) Angular	(b)	N	N		L
6. Gradation Curve Characteristics (a) Uniformly graded (b) Well graded	(b)	N	N		L
7. Soil Type (a) Sand (b) Gravel	(b)	(b)	same		V
8. Rate of Loading (a) Low (b) High	(b)	(b)	(a)		L

* - Importance of This Variable in Relation to Strength

V - Very Important
L - Low Importance
U - Unimportant
N - Not Well Defined

dominate. Lade and Duncan (Ref. 74) developed an elastoplastic static stress-strain theory for sand. The theory is applicable to general three-dimensional stress conditions, and it models several essential aspects of the soil behavior observed in experimental investigations: nonlinearity, the influence of σ_3, the influence of σ_2, stress-path dependency, shear-dilatancy effects, and coincidence of stress increment and strain increment axes at low stress levels with transition to coincidence of stress and strain increment axes at high stress levels. Results of cubical triaxial tests, torsion shear tests, and tests performed using various stress-paths were analyzed using the theory, and it was found that the stress-strain and strength characteristics observed in these tests were predicted with reasonable accuracy. The theory can be modified for repeated loading by changing the initial stress-strain conditions.

Hardin and Drnevich (Ref. 75) proposed a hyperbolic shear stress-shear-strain relation to define soil behavior during dynamic loading. Good agreement between experimental results and theory was obtained for sandy soils. Reasonably good agreement was also obtained by Seed and Idriss (Ref. 76) for plots of shear modulus as a function of log of shear strain amplitude for other sand and some gravel soils using the same equations. Knosla and Wu (Ref. 77) proposed an elastic-plastic work hardening theory to predict soil response to diverse stress paths from results of hydrostatic compression and drained compression tests, conducted under both static and cyclic loading conditions.

In general, each theory was confirmed by experimental results obtained from laboratory tests, but the generation of the stress-strain curve requires the input of the properties derived from these tests. The primary value of these relationships for this study is to explain the experimental trends rather than for incorporation in the theoretical models of the track structure.

4.2 STATIC LABORATORY PROPERTY TESTS

Descriptions of the type of tests, methods of testing, factors influencing stres-strain behavior and typical laboratory results will be briefly discussed for gravel-size particles under static loads.

Static Triaxial Tests. The triaxial compression test is used to measure the shear strength of a soil under controlled drainage conditions. Although no ASTM standard method exists, the basic components of the test remain unchanged between laboratories and can be found in any basic soil mechanics laboratory testing manual (Ref. 78, 79, and 80).

Granular soils are prepared as cylindrical specimens with a length at least twice the diameter by some method of impact or vibratory compaction in a dry to fully saturated state to a specific density. Samples encased in a rubber membrane are placed within a testing chamber and subjected to a predetermined value of confining pressure, σ_3. The sample first undergoes isotropic compression, where, if drained, a volumetric decrease occurs. A backpressure may be internally applied to the specimen to ensure saturation or simulate the in-situ pore pressure. An axial compressive stress termed the deviator stress, $\sigma_1 - \sigma_3$, is applied to the specimen at strain rates of 1/4% to 2% per minute (Ref. 78) until failure occurs. The peak deviator stress is used to determine the major principal stress, σ_1. Three or more tests are performed at different confining pressures to define the Mohr's failure envelope in terms of effective stresses. This envelope is usually in the form of a straight line passing through the origin from which the slope determines the angle of shearing resistance, ϕ, (Fig. 4.3).

The following factors have been shown to affect triaxial compression test results: maximum particle size, particle shape and surface characteristics, confining pressure, density or void ratio, soil gradation, amount and plasticity of fines, and specimen diameter to maximum particle diameter.

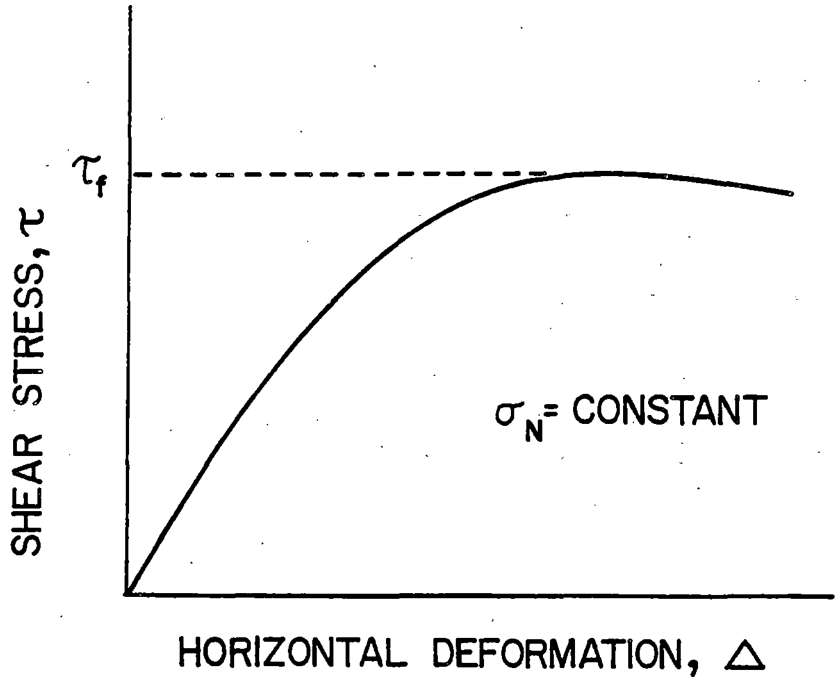
Leps (Ref. 81) reviewed and compiled triaxial compression test data for rockfill materials. Correlation of angle of shearing resistance, ϕ, to normal pressure on the failure plane, σ_N, showed that the higher ϕ values at a given σ_N are associated with high density, well-graded and strong particles, while lower ϕ values correspond to low density, poorly-graded, and weak particles. The effects of relative density, gradation, particle shape, crushing strength, and degree of saturation could not be quantitatively isolated.

A listing of references containing property data on gravel and rockfill materials from static triaxial tests include: Chen (82), Zeller and Wulliman (83), Lewis (84), Lowe (85), Fukuoka (86), Townsend and Madill (87), Marachi, et al. (68), Becker, et al. (88), Gray (89), Yoder and Lowrie (90), Schultze (91), Ferguson and Hoover (92), Banks and MacIver (93), Sowers (94), Holtz and Gibbs (95), Leslie (96), Marsal (97), Lee and Farhoomand (98), Marsal (99), Marachi, et al. (100), Fumagalli (101), Glynn (102), Boughton (103), Al-Hussaini (104), Kalcheff (66), Holtz and Ellis (105), Huang, et al. (106), National Slag Association (107), and Jones, et al. (108). For ballast materials the references are: Raymond, et al. (55), Chung (109), Raymond, et al. (42), Olowokere (110), and Rostler, et al. (111).

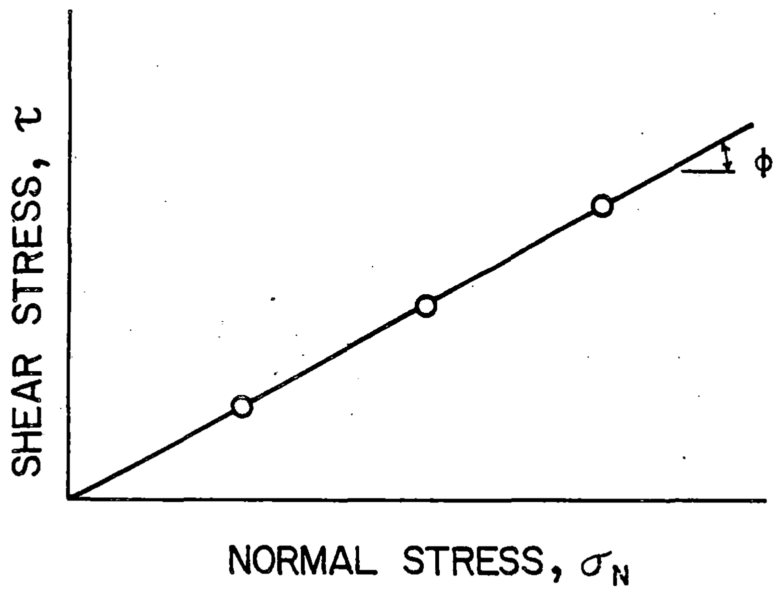
<u>Static Direct Shear Test</u>. The direct shear test is a shear strength test under drained conditions, in which the specimen is forced to failure on a

predetermind plane(s). An ASTM standard method for consolidated-drained conditions in single and double shear, ASTM D3080, is available. Details of the test procedures can also be found in soil mechanics laboratory manuals. Other investigators have modified testing equipment to suit specific needs of loading and drainage conditions; however, the test basically remains the same. The specimen is prepared wet or dry to a specific density, in a rigid circular or annular ring or in a square box, by vibration or compaction techniques. The function of the rigid container is to permit no lateral deformation of the specimen. Thus the frictional resistance developed is through particle interactions which will yield the angle of shearing resistance. A predetermined normal load is applied to the top of the specimen at which time the shear box may be filled to saturate the specimen. The specimen is allowed to drain and consolidate under the desired normal force or increments thereof prior to shearing. Backpressures cannot be applied, nor can pore water pressures be measured during testing. A horizontal force, i.e., the shearing force, is applied to the top of the shear box at a controlled rate of deformation, or it can be applied at a constant strain rate which is the same as that stated for triaxial compressions tests. The peak horizontal stress is taken as the failure shear stress, and the failure envelope for three or more tests performed at different normal loads defines the angle of shearing resistance (Fig. 4.7).

Taylor (Ref. 112) and Rowe (Ref. 113) reported values of shearing resistance, ϕ, for sands to be about 1° to 2° larger from the direct shear test than from the triaxial compression test. These are small differences from an engineering viewpoint, and the results are essentially the same. Such comparisons for gravels have not been made. It has also been noted that



a) SHEAR STRESS vs. DEFORMATION CURVE



b) FAILURE ENVELOPE

Figure 4.7. Typical Plots of Direct Shear Test Results

the stiffness and Poisson's ratio cannot be determined from the direct shear test.

The following factors have been shown to affect direct shear test results: the change in the area of the sample with shear deformation; the possibility that the actual shear failure surface is not plane, and not necessarily the weakest one; nonuniformity of stresses over the shear surface; the ratio of specimen thickness to maximum particle size; and the same factors which affect triaxial test results with the exception of confining pressure which should be replaced by normal load.

The references containing direct shear property data for gravels and rockfill materials are: Lewis (Ref. 84), Fukuoka (86), Hennes (114), Sowers (94), Wilson and Marand (115), Bishop (116), and Boughton (103). This test has not been performed on ballast materials.

One-Dimensional Compression Test. The one-dimensional compression test (oedometer test) is concerned with the deformation response in the axial direction while the specimen is restrained against lateral strain. This test is more commonly known as the consolidation test, in which the pore water pressure of saturated sample is allowed to dissipate under the applied load. However, due to the high permeability of granular soils, there is essentially no pore pressure development; and thus, the load is immediately transferred to the soil particles. The ASTM standard for this test is D2435-70, and similar procedures can be found in soil mechanics laboratory manuals. Specimens are usually prepared by compaction or vibration techniques within a rigid circular container. An axial load is applied to the specimen, and the deformation recorded is the total deformation which remains constant under the applied load. The axial load is increased in increments and the deformations recorded in a similar manner until the maximum load is reached. A failure condition is not reached in this type of test, but a stress-strain curve is obtained such as shown in Fig. 4.6a.

Since this test permits no lateral strain of the specimen, several investigators have devised test equipment which can measure the horizontal stress. This corresponds to the at-rest state, which defines the coefficient of at-rest pressure, K_o, as the ratio of the effective horizontal stress to the effective vertical stress. This coefficient was theoretically related to the peak value of the angle of shearing resistance, ϕ, by Jaky (Ref. 117) as

$$K_o = 1 - \sin \phi \quad . \tag{4-1}$$

This equation reasonably approximates experimental results for sands investigated by Moore (Ref. 118), Hendron (Ref. 119), and Al-Hussani (Ref. 120).

The factors affecting oedometer test results are: side friction which permits a non-uniform distribution of stresses within the specimen, thus preventing a one-dimensional state of strain; particle crushing; specimen thickness to maximum particle diameter; and the same factors which affect direct shear test results.

In order to define a general stress-strain relationship from experimental results, property data from the following references on gravels or rockfill materials should provide useful information: Fumagalli (Ref. 101), Murdock (121), Kjaernsli and Sande (122), Boughton (103), Sowers, et al. (123), Pigeon (124) and Bernell (125).

4.3 MODELING TECHNIQUES

The size of conventional triaxial, oedometer and direct shear test equipment has prohibited the testing of rock or large gravel size particles often used in rockfill dams. Some investigators have constructed apparatus capable of testing this full size field material. However, the testing equipment costs and sample preparation time makes this approach undesirable. A number of methods have been developed by which the particle sizes of the field material, termed the prototype, is scaled downward to sizes easily accommodated by available test equipment, termed the model. These modeling techniques are an attempt to duplicate the behavior of the prototype material.

The modeled material must conform to certain fundamental requirements, such as density, particle shape, relative grading and mineralogy. Discretion must be used in applying results for material not satisfying the preceding criteria. The modeling methods developed have been applied to a limited number of cases, so their general validity is not yet determined.

The various modeling methods will be described and subsequently evaluated to determine the most suitable method. The reliability of the strength test results is difficult to assess, since one criterion for the rejection of a method would be a field failure of the prototype material. This event has not occurred in any of the past experience.

Zeller and Wullimann (Ref. 83) developed the scalping technique for the Goehenenalp Dam shell material. Scalping employs the removal of the coarser prototype material to obtain different grain size distributions (Fig. 4.8). Each grain size curve contained the same percentage of quartz, feldspar and mica, and the particles were subangular to angular in shape. Results from triaxial compression tests were not influenced, provided that the specimen diameter was at least five times the maximum grain size. The shear strength of the shell material is obtained by extrapolation of the shear strength versus porosity curves for the maximum particle diameter of each gradation curve (Fig. 4.9).

Lewis (Ref. 84) performed drained direct shear tests on different

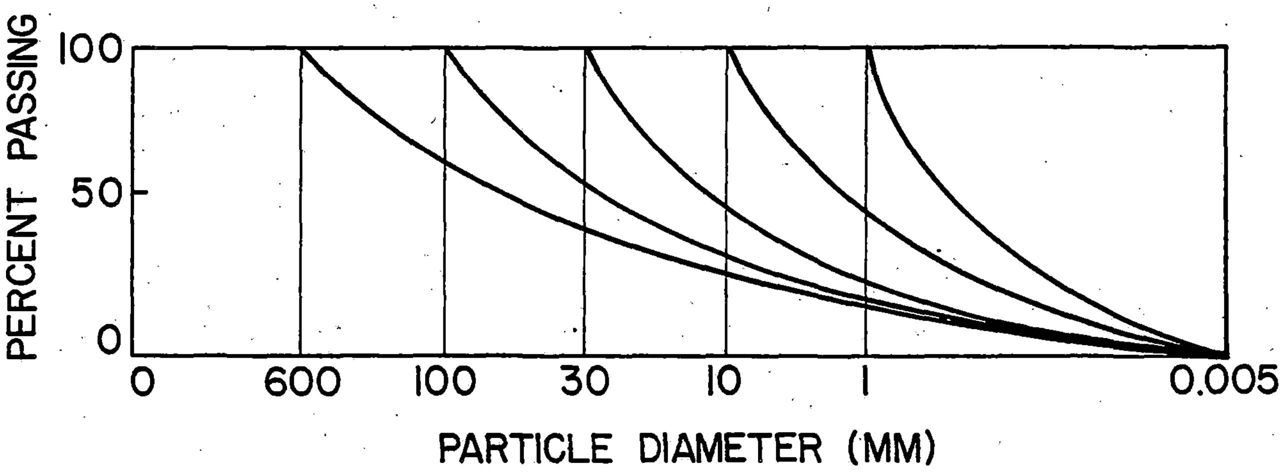


Figure 4.8. Gradation Curves by the Scalping Method (Ref. 83)

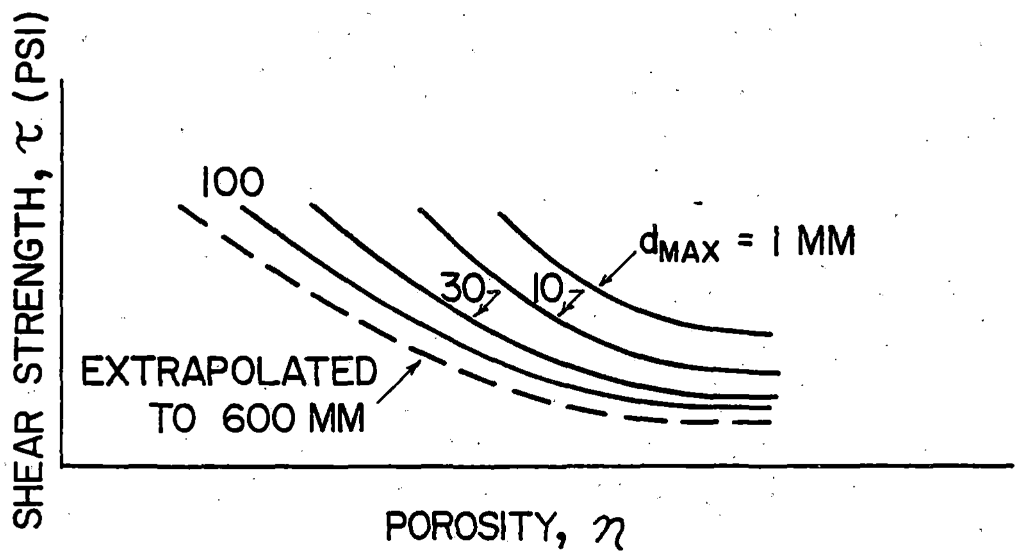


Figure 4.9. Shearing Strength as a Function of Porosity and Grain Size Distribution (Ref. 83)

sizes of crushed granite having one size stone for each grading. The modeled material maintained the same density, particle shape and relative grading of the full scale materials. The author obtained a linear relationship between effective angle of internal friction and log of particle size for 6 cm square and 12 in. square shear boxes (Fig. 4.10). These results were valid for particle sizes not exceeding 1/40th the width of the shear box. The friction angles for larger grain sizes are obtained by extrapolating this straight line.

Fukuoka (Ref. 86) attempted to establish relationships between gradation curves of materials and the physical properties of soils by using Talbot's equation for an ideally graded material. Talbot's equation is

$$p = 100 \left(\frac{d}{D} \right)^{n} , \qquad (4\text{-}2)$$

in which d is the diameter of the sieve, p is the percent passing a given d, D is the maximum particle diameter, and n is an index varying from 0.15 to 0.55. The objective of this study was to find the n value yielding the most desirable properties of soils used as construction materials. The author was not directly concerned with scaling down the sizes of the prototype material, but rather, determining the best grading of the field material based upon strength characteristics. Thus, the field material can be screened and processed with the expectation that the behavior from laboratory tests will be duplicated in the field. This method can be utilized for the purposes of scaling by performing strength tests on soils having different values of D and n. The results can then be extrapolated to the prototype material which conform to the proper grading from laboratory tests (Fig. 4.11). For these soils, as those tested by Fukuoka (Ref. 86), the specimen diameter

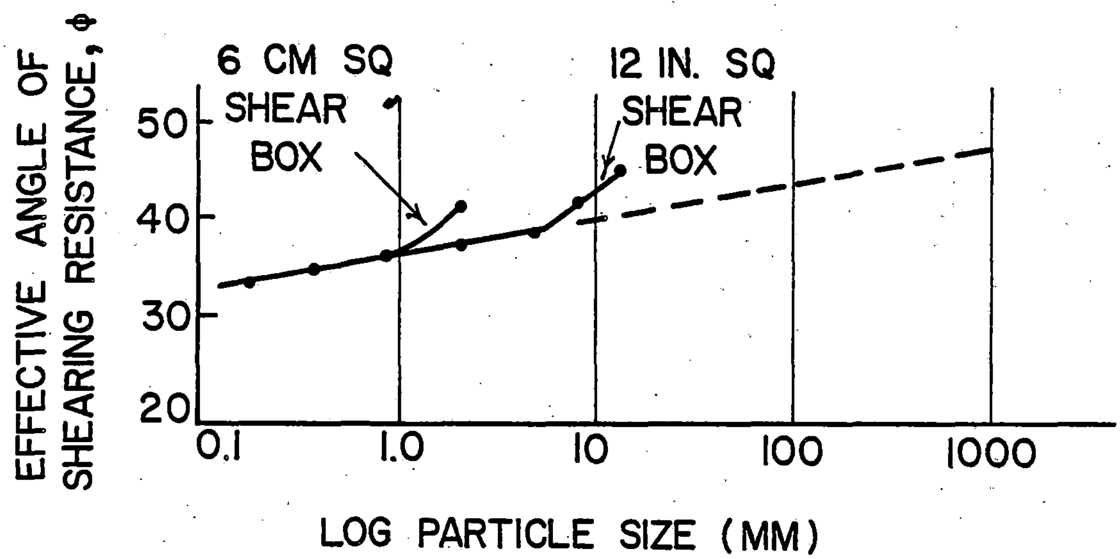


Figure 4.10. Angle of Shearing Resistance for Uniformly Graded Crushed Granite as a Function of Particle Size (Ref. 84)

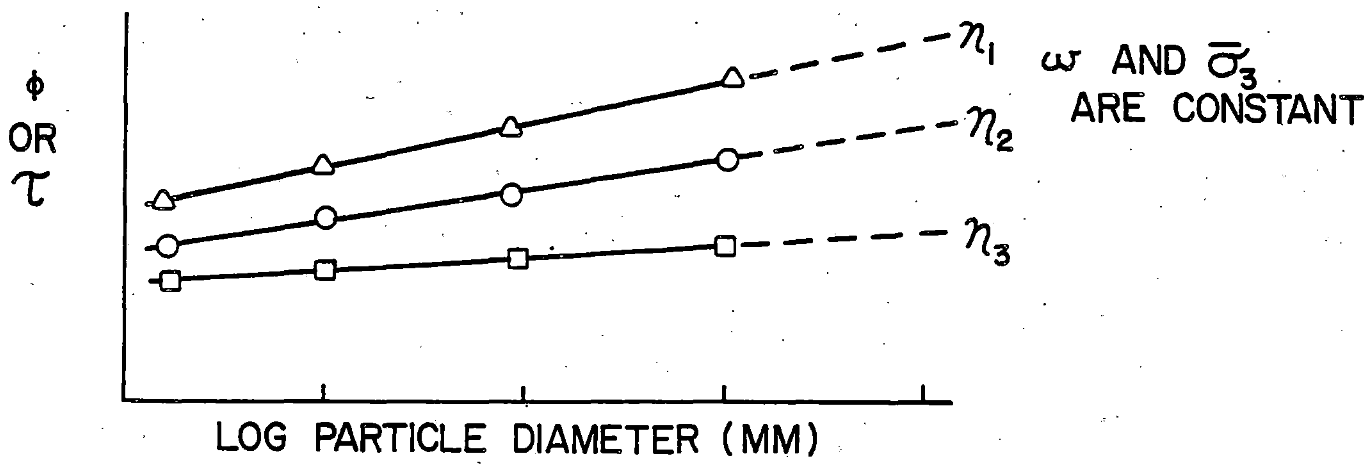


Figure 4.11. Illustration of Variation of Strength Parameters with Particle Diameter and Grading Index (n)

must be at least six times the maximum particle diameter for the well-graded materials.

Lowe (Ref. 85) modeled a well-graded river cobble gravel and a skip-graded terrace cobble gravel to be used for the shell of an embankment by the parallel grading method. The modeled gradation curve was 1/8 the size of the prototype material (Fig. 4.12) and the particles duplicate the shape and mineralogical composition of the field material. The author justifies the use of the parallel grading method based upon theoretically derived equations for uniformly packed elastic spheres. The contact stresses and strains are functions of the applied external stress and the modulus of elasticity of the sphere material. The size of the spheres did not affect the contact stresses and strains; therefore, the behavior of the model and prototype material should be similar. Samples were prepared at void ratios expected in the prototype material and tested in triaxial compression with the specimen diameter four times the maximum particle size. For design purposes, the angle of internal friction angle of the prototype river cobble gravel was taken 2^o to 3^o less than the friction angle of the modeled material, whereas the terrace cobble gravel friction angle is assumed to be equal for model and prototype, since the author did not state otherwise.

Fumagalli (Ref. 101) states that reduced scale tests can be simulated when the following requirements are met: 1) similarity of the gradation curve, 2) same initial void ratio, and 3) the same shape coefficient of the material. The shape coefficient is the ratio of the volume of the granules to the volume of the spheres circumscribing them. The quadratic grain size curve is used for modeling and is expressed by

$$p = 100 \quad \frac{D}{D_{max}} \quad , \qquad (4\text{-}3)$$

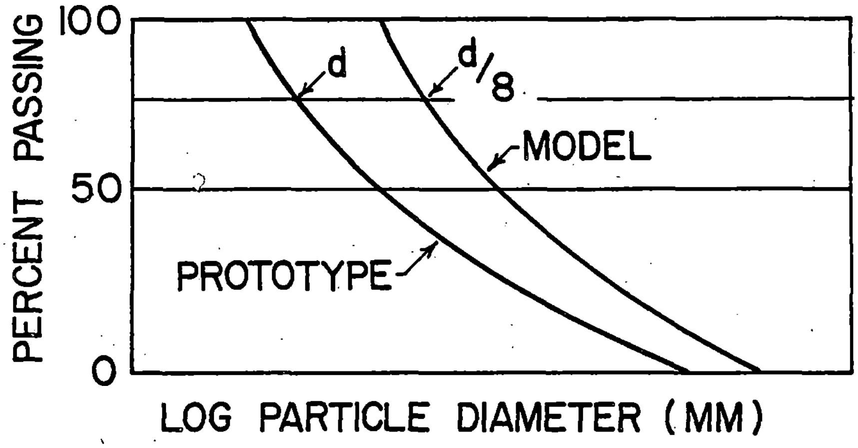


Figure 4.12. Gradation Curves by Parallel Grading Method (Ref. 85)

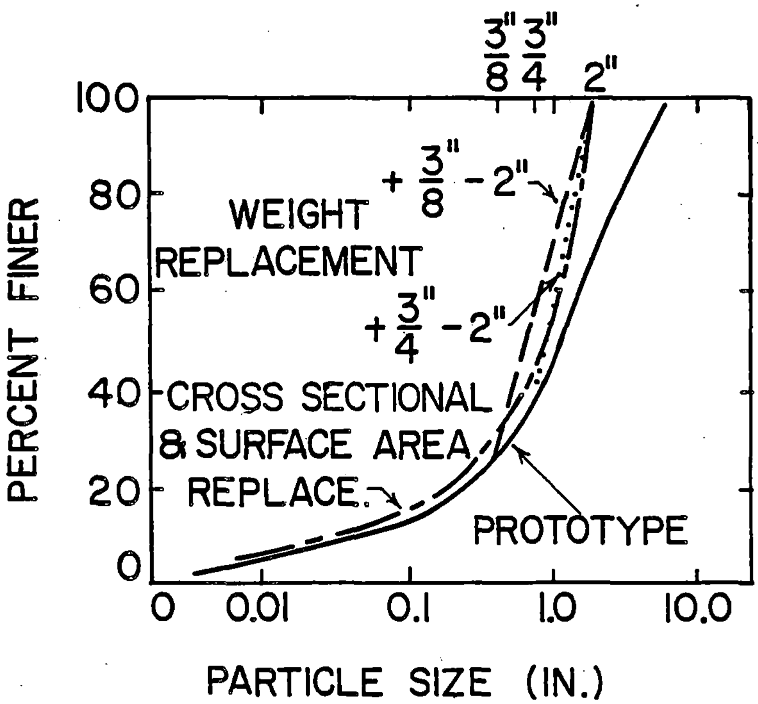


Figure 4.13. Gradation Curves by Replacement Methods (Ref. 126)

in which D is the particle diameter, p is the percent in weight passing for a given D, and D_{max} is the maximum particle diameter. The author does not correlate prototype to model gradation curves, but compares results for various size modeled materials using testing equipment with different diameters. The implication is to grade the field material by the laboratory gradation curve yielding the most favorable behavioral characteristics. Results from tests performed indicate the ratio of the maximum particle diameter to the diameter of the specimen must be equal to or less than 1/5 for well-graded material and less than 1/5 for monogranular material.

Frost (Refs. 126 and 127) proposed modeling by the replacement method. The coarse fraction, which cannot be prepared in the testing equipment, is replaced by the finer fraction satisfying either of the following: 1) the same weight, 2) the same cross-sectional area, 3) the same surface area, or 4) the same volume as the coarse oversize fraction omitted. In all cases, the maximum and minimum particle sizes of the replacement fraction are selected. The author uses 1 in. or 2 in., and 3/4 in., 3/8 in., or the number 4 sieve, respectively. The replaced and replacement material both have the same minimum particle size.

The first case is similar to the weight replacement method used in laboratory compaction tests. The coarse fraction between the maximum particle diameter of the prototype material and the minimum particle size of the replacement material is replaced by weight with the coarse fraction of the selected maximum and minimum particle sizes of the replacement material (Fig. 4.13). For cases 2 and 3, a point for the model gradation curve is obtained from the equation,

$$p = \frac{d_{ave}}{D_{ave}} \frac{G_s}{g_s} P \quad , \tag{4-4}$$

in which p is the percentage of the replacement fraction by weight, d_{ave} is the average particle size of the replacement fraction, D_{ave} is the average particle size of the replaced fraction, G_s is the specific gravity of the replaced fraction, g_s is the specific gravity of the replacement fraction and P is the percentage of the replaced fraction by weight. The model gradation curve is drawn through three known points: the maximum particle size of the model material with 100% finer than by weight, the p value at d_{ave}, and at the particle diameter of the prototype material with 0% passing (Fig. 4.13). The 4th case utilizes a method similar to cases 2 and 3; however, the equation is of the form,

$$p = \frac{g_s}{G_s} P \qquad (4\text{-}5)$$

Both equation (4-4) and (4-5) are derived from geometry with the assumption that the replacement fraction has similar degrees of lack of sphericity as the replaced fraction. The author does not present shear strength test results, but indicates the cross sectional area replacement method will more closely duplicate the expected laboratory maximum dry density of the prototype material.

The modeling techniques which have been presented and discussed are: scalping, trial and error method of maximum particle size to size of the test specimen, Talbot's equation, parallel grading, square root method, and the weight or volume replacement methods. The parallel grading method is best suited for modeling because: 1) the method is theoretically justified, 2) obtaining and preparing the gradation curve of the modelled material is relatively easy, 3) the field material can be used, instead of being screened and processed as implied from Talbot's equation and the square root method,

4) the strength results of the prototype do not have to be obtained by the extrapolation techniques used for the scalping and the ratio of particle size to test specimen size methods, and 5) other researchers, such as Marachi, et al. (Ref. 68.), Becker, et al. (Ref. 88.), and Chen (Ref. 82), have used this method with much success.

4.4 FIELD METHODS

The most widely used field methods for granular soil property determination are those for penetration resistance, California Bearing Ratio (CBR), plate bearing resistance, and seismic velocity. The penetration devices and the CBR test are not suitable for ballast measurement, however, because of the large particle size in relation to the equipment dimensions. Thus only the plate and seismic tests will be discussed.

Plate Bearing Test. Several types of plate bearing test have been devised to evaluate the strength characteristics of gravelly soils. Two test procedures have been standardized by ASTM, one for nonrepetitive static tests (ASTM D1196) and the other for repetitive static plate load tests (ASTM D1195). A pyramid of rigid steel plates ranging from 6 in. to 30 in. in diameter is loaded by a calibrated hydraulic jack. The test is stress controlled. The plate deflection at various loads is measured and the modulus of subgrade reaction, k, is computed by dividing the load by the plate area and by the deflection.

Factors which affect plate test results are plate diameter, plate shape, loading rate, maximum grain diameter, size of test pit, ground water level, overburden pressure, and method of seating plate on the test surface.

McLeod (Ref. 128) performed repetitive plate bearing tests on gravel base course and subgrade materials at ten airport runways which have been in service for several years. Plate diameters of 12, 18, 24, 36, and 42 in. were used, and compared to data obtained by the standard 30 in. diameter plate. A linear relationship between unit load and the perimeter area ratio was obtained at different deflections for each test section.

Zimpfer (Ref. 129) investigated the effects of plate radius to base course thickness with plates having 1.95, 4, 8, and 12 in. diameters on a 4 to 11 in. thick limerock gravel base material over a clay-sand subbase. Test results indicate that a minimum base thickness of 1.5 times the diameter of the loaded plate is necessary for calculation of the modulus values. Fair agreement is obtained between Burmister's two-layered system theory and experimental results. Thus, the authors think the use of layered theory is quite promising. Young's modulus values were used, and are calculated by knowing the subgrade modulus, k, at the desired deflection. The k values for the base material varied from 10.4×10^3 pci for 1.95 in. diameter plate to about 1.7×10^3 pci for 12 in. diameter plates at 0.1 in. deflection.

Meigh and Nixon (Ref. 130) performed plate bearing tests using 18, 26, and 36 in. diameter plates and 12 in. square plates.

Rodin (Ref. 131) compared results of plate bearing tests with 18 in. and 30 in. diameters, and 12 in. square plates. Testing was stress controlled with load increments of 15 minutes in sandy gravel and gravel deposits at various depths.

Dvorak (Ref. 132) examined the deformation properties of sandy gravels, gravelly sands and clayey silty gravels in-situ by means of loading tests. From a great number of tests the authors were able to draw the conclusion that for a well-graded gravel sand, the maximum particle diameter of a grain, d_{max}, may be as large as 100 to 150 mm for a plate area of 0.1 m^2 (14 in.

diameter); i.e., d_{max} = 0.3 to 0.4 times the plate diameter.

Nielsen and Lowe (Ref. 133) conducted a laboratory investigation with plate load tests to determine the modulus of elasticity of a select base and crushed rock base course materials for 2 and 3 layered systems on a mechanical subgrade. The device simulates the action of a natural subgrade and provides a yielding support for pavement studies. Rigid bearing plates 30, 24, 15, and 8 in. in diameter were used, and tests performed in accordance with ASTM D-1195-57. All four bearing tests were conducted on the same base. Each plate was subjected to three complete load cycles. The authors believe that the third loading cycle is representative of the true load-deflection relationships of the base material, and should be used to determine the modulus of elasticity and Poisson's ratio.

Selig (Ref. 134) also conducted 6 in. diameter plate load tests on base course materials as a means of measuring the effects of field compaction. Details of the apparatus and procedure are described in (Ref. 135).

Plate bearing tests on ballast materials have been performed by McLean, et al. (Ref. 136), Peckover (Ref. 137), and Prause (Ref. 138). The test results are discussed in Ref. 23.

Seismic Velocity. Dynamic disturbances, such as a hammer blow, applied to the ground surface will generate three types of waves: compression, shear, and Rayleigh. A description of these waves and their measurements are given in Ref. 139.

The compression wave has the highest velocity so it can be measured by observing the travel time of the fastest surface disturbance. From this measurement the Young's modulus of elasticity can be calculated if the mass density and Poisson's ratio are known or estimated. The Rayleigh wave is a surface wave that travels with a velocity very nearly equal to the shear wave velocity. Its velocity is measured by applying steady-state vibration to the surface and determining the wave length of the surface wave. From this Rayleigh wave measurement the shear modulus can be calculated if the mass density is known. Young's modulus can be calculated from shear modulus if Poisson's ratio is assumed. However, vibration affects the physical state of ballast so this technique may not be suitable.

The seismic method of testing offers the following advantages: 1) the dynamic moduli can be determined under the in-situ state of stress, 2) large areas can be covered, and thus a better average of the layer properties may be determined, and 3) a site may be covered quickly. However, because the tests are conducted using very low stress level disturbances, the resulting moduli are probably much larger than the moduli representing the material behavior in a track structure under train loading. Factors influencing test results are material type, degree of compactness of the material, moisture conditions and test procedures.

Heukelom and Foster (Ref. 140) used high frequency (10,000 Hz) and low frequency (600 Hz) steady state vibrations to obtain the stiffness, and hence wave velocity values of base course materials from which the dynamic modulus of elasticity was determined. The wave velocity in homogeneous clay gravel ranged from 190 to 320 m/sec and the resulting Young's modulus was 2100 to 6000 kg/cm^2. Bergstrom (Ref. 141) demonstrated agreement between the dynamic Young's modulus derived from velocity measurements on homogeneous soil with the moduli from bearing tests.

Selig (Ref. 134) used seismic compression wave velocity measurement as a means of evaluating the factors influencing compaction of base course materials. Details of the apparatus and procedure are described in Ref. 135. Seismic velocity was shown to increase with number of compactor coverages and thickness of base course materials. The soil type also was a significant factor, with the clean limestone yielding the highest seismic velocity. Seismic velocity values varied from 420 to 780 ft/sec for the base course materials tested. No meaningful correlations could be established between seismic velocity and the other strength tests or wet density. More research was shown to be needed on the relationship of the compression wave velocity to the material physical state conditions.

Weaver and Rebull (Ref. 142) applied seismic techniques to the measurement of soil compaction for construction control on eight sandy-gravel embankment projects. No two of the eight soils exhibited similar relationships between compression wave velocity and either wet, dry, or percent maximum density. Values of wet density, however, produced the least scatter in correlation with velocity. However, a larger degree of scatter would occur for the computed Young's modulus of elasticity, since this modulus is proportioned to the velocity squared. The researchers concluded that direct correlations between velocity and soil density 1) have no linear relationship individually, 2) have no useful relationship by soil group, and 3) show a scatter far beyond that expected on the basis of individual test repeatability, thus making their use impractical for compaction control.

4.5 DYNAMIC PROPERTY TESTS

The category of dynamic test considered in this chapter involves some type of repeated load or cyclic load. The other category of dynamic test concerns the effect of loading or strain rate during a single load application. However this type of test has not been commonly applied to granular materials. Factors influencing dynamic test results include shear strain amplitude, void ratio, number of cycles of loading, effective mean principal stress, degree of saturation and stress history.

Cyclic simple shear apparatus suitable for testing coarse dry granular material was developed by Ansell and Brown (Ref. 143). Both volumetric and shear strains could be measured as well as the normal and shear stresses. The device consists of a rectangular box with ends that rotate to shear the sample. The sample size is 210 mm x 140 mm by 30 mm deep. A description of the apparatus and an evaluation of its suitability are given in the reference. A brief review of other, previously used, approaches is also given.

Sparrow (Ref. 144) developed a repeated load biaxial shear device for testing ballast in a manner intended to simulate the stress conditions under a tie during operations like tamping and traffic. The device contains a 10 cm cube specimen. The four vertical boundaries are interleaved to permit independent horizontal movement with large deformation. Repeated loading is applied with pneumatic actuators with each of the two horizontal axes independently controlled. The horizontal top and bottom boundaries have a constant force applied, but movement must also be restricted to clear the moving sides of the box. Thus the cyclic loading and deformation is essentially biaxial. Phillips (Ref. 145) obtained good correlation in trends between the permanent strain results with this biaxial box and repeated load triaxial test on similar ballast material.

Another type of cyclic simple shear test uses a wire-reinforced membrane to confine the soil. The sample is subjected to an applied vertical load and deformed in shear at frequencies on the order of 1 to 2 Hz. However, available apparatus of this type is not suitable for coarse materials such as ballast.

In the resonant column test, solid or hollow cylindrical specimens in an isotropic state of stress are excited at the top or bottom in either the longitudinal or torsional modes of vibration. The frequency of loading ranges from 20 to 100's of Hz. The sample strains in this test are generally much smaller than those in the triaxial or simple shear test. Because of the vibration sensitivity of ballast, this test may also not be suitable for use with ballast. Also apparatus used to date (1978) will not permit large enough sample size.

Wong, Seed and Chan (Ref. 146) used cyclic triaxial test to study the strength loss in saturated, undrained samples of gravel soils. Their apparatus provided the stress state shown in Fig. 4.6c by applying an alternating tension-compression axial load(deviator stress) to a hydrostatic stress state.

Both triaxial compression tests and one-dimensional compression tests are performed with the type of loading shown in Fig. 4.6d. These tests, designated repeated load tests, are usually stress controlled. Repeated loads start from zero and are increased to some predetermined magnitude and then decreased to zero again, thus never putting the sample into axial extension. The process is repeated until either the desired number of cycles or a limiting permanent strain is reached.

Repeated load triaxial tests on gravels have been reported by Bamert, Schnitter and Weber (Ref. 147), Hicks and Monismith (148), Hicks (149), Dunlap (150, 151, 152), Seed et al. (153), Barksdale (154), Wolfskill (155), and Allen (156). One-dimensional compression tests on gravel have been reported by Chen (82) and Schultze and Coesfeld (157). Repeated load triaxial compression tests on ballast materials were performed by Chung (109), Raymond, Gaskin and Svec (42), Olowokere (110), Thompson (158), Knutson, et al. (159), and Rostler, et al. (111). One-dimensional compression tests on ballast materials are presented by Bishop (160), Powell (161), and Wong (162).

The in-situ ballast behavior associated with in-service traffic loads is currently most easily simulated in the laboratory by the cyclic and repeated load triaxial tests. These tests conceivably have the capacity to represent the stress conditions imposed upon the loaded in-situ ballast material. Assuming that the in-situ state of stress and ballast physical state are appropriately defined, then the resilient (elastic) and the residual (permanent) ballast properties are easily derived. The parameters usually determined are the resilient strain, the residual strain, the resilient Poisson's ratio, and the resilient modulus (repeated deviator stress/resilient axial strain). A relationship for each parameter with cycles of loading can be established. However, the resilient modulus and the permanent strain or deformation are the parameters receiving the most attention.

A recent and extensive review of previous research on granular materials by Knutson, et al. (Ref. 159) and by Thompson (Ref. 158) provided a basis for an experimental program to investigate the variables most likely to influence the behavior of several ballast materials. These studies confirmed past findings that identified stress level as the most important variable affecting the resilient modulus (E_r). That is, the E_r value would increase as stress level increased. For open-graded ballast materials, the resilient modulus was shown to be only slightly affected by material type and, virtually independent of changes in gradation, stress history, and density.

Stress level, number of load applications, and degree of compaction, i.e., initial density, were also confirmed by Knutson, et al. (159) to be the most impportant variables affecting permanent deformation of ballast materials. In general, the cummulative permanent deformation increases with increasing cycles of loading and is much greater for specimens prepared at a loose initial density state than at a dense state. Permanent deformation was also shown to be slightly influenced by differences in gradation and in particle shape.

Particle breakdown or change in the initial gradation occurs after many applications of repeated load or under the application of high confining stresses. This effect becomes more pronounced with angular particles than rounded particles because of the higher particle contact stresses. Well-graded materials also experience less breakdown than uniformly-graded materials, since more points of particle contact are present. Individual particle strength is another factor influencing the degree of crushing. The amount of sample degradation may be defined in terms of the particle breakage factor, "B," defined by Marsal (Ref. 97). Examples of particle degradation are shown in Dunlap (Ref. 151), Wolfskill (155), and Chen (82) for gravel soils, and Olowokere (110), Chung (109), and Bishop (160) for ballast materials.

5. COMPACTION OF GRANULAR MATERIALS

The degree of compaction is one of the most important parameters influencing the mechanical properties and engineering performance of the soil. Especially in granular or cohesionless materials, the density has been generally used as a descriptive parameter of physical state for various aspects of soil behavior. Compaction is the volume reduction of soil by means of mechanical manipulation. It involves the reduction in air voids through particle rearrangement, while the water content of the soil generally remains constant. Consequently, the degree of saturation increases during compaction. Since compaction does not involve expulsion of pore water, it can be accomplished relatively quickly. Thus, it should be distinguished from consolidation which is the gradual volume reduction process through expulsion of pore water in fine-grained soils. It should also be distinguished from tamping which is a process of particle rearrangement with mechanical stirring or pounding action, causing either densification or loosening. The above three terms are often used interchangeably in the railroad industry in relation to densification of ballast.

The purpose of compaction is to impart to a soil desired properties it does not have in its existing state, or to improve the physical performance of soils. In general practice of geotechnical engineering, compaction is performed for purposes such as 1) to increase strength, 2) to reduce compressibility, 3) to minimize the volume change potential, 4) to decrease permeability, 5) to control resiliency properties, and 6) to reduce frost susceptibility.

To be effectively used, compaction has to be properly applied considering the purposes to be achieved and the compaction conditions given, such as soil type, moisture condition, and the present and subsequent changes of environmental conditions.

This chapter summarizes the basic principles of compaction, with emphasis on granular soils, that is coarse-grained soils that have little cohesion, like sand and gravel. Effects of compaction on the mechanical properties of granular materials are reviewed, and various methods of laboratory compaction are described. Special attention is given to vibratory compaction. Highway related field compaction methods and compaction control are briefly introduced, and then various aspects of the ballast compaction applications are discussed.

5.1 LABORATORY COMPACTION TESTS

Laboratory compaction tests involve applying a certain amount of compactive effort to soil samples in a container of a given volume. Since the first introduction of the compaction tests by Proctor (Ref. 163), various methods of laboratory compaction have been proposed and used 1) as means of determining proper compaction to be achieved in the field, and 2) as a research tool for investigating compaction behavior of the soil in the laboratory.

The principal types of laboratory compaction effort are the impact type, the kneading type, the vibratory type, the static type, and the gyratory type. Among the reasons for the different compaction efforts are 1) to accommodate the fact that one soil is different from another in its response to compaction, and 2) to simulate the field compaction behavior with different rollers as closely as possible. A summary of the methods and the factors influencing the results are given in Ref. 164.

Impact Compaction. The impact compaction method is the most widely used of the laboratory methods. Soil is compacted with the force generated from a hammer of specified weight and shape freely falling from a specified height. The variations in the method are a function of the weight of the hammer, height of the fall, dimensions of the mold, number of soil layers, and maximum particle size material being compacted. The most commonly used impact methods are 1) ASTM D-698 or its AASHTO equivalent T-99 which uses about 12,400 ft-lb/cu ft effort, and 2) ASTM D-1557 or AASHTO T - 180 which uses 56,000 ft-lb/cu ft effort. A disadvantage of the method is that it applies the compactive effort in a manner that is quite different from field methods. In the case of ballast, another serious disadvantage is the particle breakage caused by the impact of the steel hammer.

Typical laboratory test results for two different sizes of graded crushed limestone are shown in Fig. 5.1 for three different impact compactive efforts (Ref. 165). The dry density achieved is highest when the material is close to or at saturation, and somewhat lower when compacted dry. The lowest densities occur for compaction at moisture contents between the dry and saturated states. Such a trend is characteristic of compaction behavior of granular materials, especially sands. It is partially due to the capillary stresses developed in the partially saturated soil which tend to resist the compactive effort. Free draining materials, for example open-graded crushed rocks, generally either exhibit no distinct optimum at any moisture content or yield the highest dry unit weight at the maximum moisture content that can be used, i.e., when saturated (Refs. 165 and 166).

Many types of equipment and procedures for both field and laboratory compaction will produce some breakage of aggregate particles, particularly with uniform gradation and high angularity like ballast. However, insufficient data are available for correlation of degradation with various compaction procedures and compactive efforts. Increase in compactive effort in general results in an increase in degradation. The rate of degradation is high during the initial part of the application of compactive effort, and thereafter

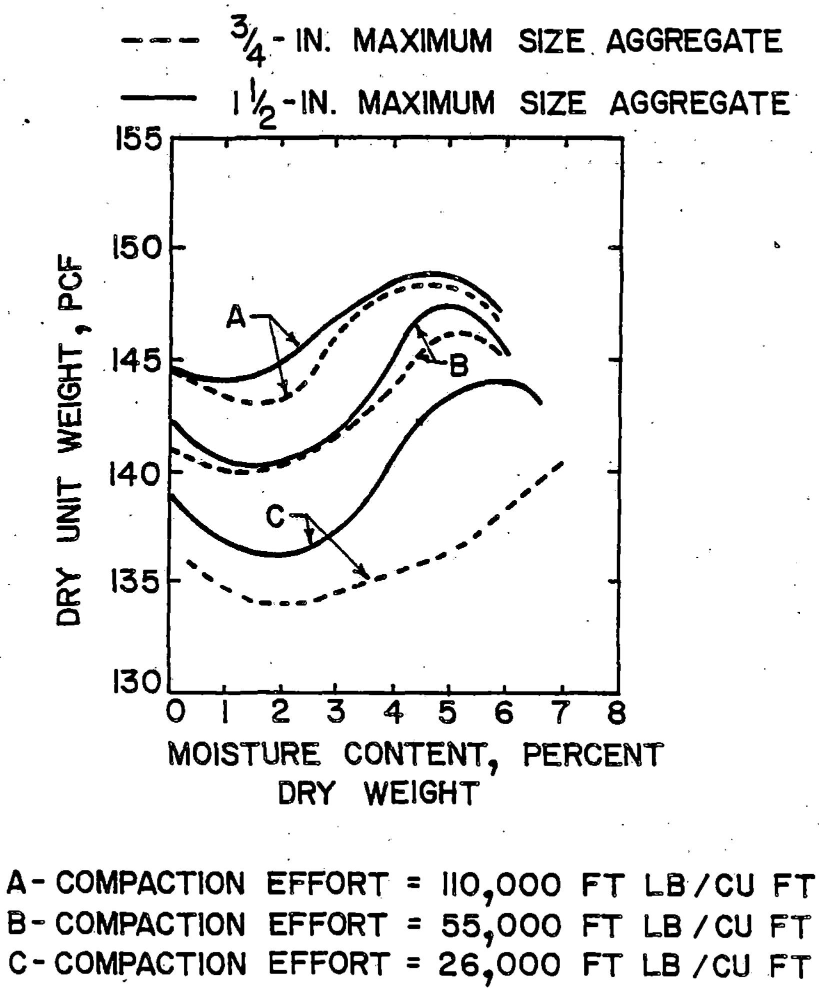


Figure 5.1. Laboratory Compaction of Graded Crushed Limestone under Various Compaction Efforts (Ref. 165)

becomes less as the compactive effort is increased. Degradation during compaction not only influences maximum dry unit weight directly by producing a more widely-graded particle-size distribution but, it also results in a change in specific gravity of the total aggregate. This is the result of exposing a greater number of previously impervious voids in the individual particles and thus increasing the apparent specific gravity as the particle size is reduced.

Other Methods. The kneading method was developed in an effort to devise a method of laboratory compaction that would more closely simulate field compaction. It was recognized that rubber-tire and sheepsfoot rollers apply pressure briefly with little or no impact but with some kneading or shearing action in the soil. With the kneading method, a compaction foot is used that is much smaller in area than the exposed soil sample surface area. The foot pressure built up gradually, allowed to dwell on the sample for a specified duration of time, and then again gradually released. An important aspect of kneading compaction is the development of lateral shearing stresses and strains which appear to be analogous to those developed during compaction by a sheepsfoot roller.

Parameters defining the compaction effort of kneading compactors are peak foot pressure, cycle time, dwell time, size of mold, number of soil layers, and number of applications per layer. Although the Harvard miniature compactor which employs kneading compaction techniques is frequently used (Ref. 167), there is presently no standard test method for determining maximum dry density and optimum water content that applies to kneading-type compactors. However, two AASHTO standards using kneading compaction (T-173 and T-175) are available for compacting soil specimens for other purposes. Details of this test apparatus as well as other kneading compactors can be found in Ref. 164.

The static compaction method involves slow application of pressure to most or all of the exposed soil surface and holding it for specified time. This method was used only to a limited extent in early soil compaction testing. Now, it is rarely used except for comparison with other methods or for preparation of soil samples for property tests. It is expected to be ineffective in compacting ballast. Surface area of pressure application, rate of increase, dwell time, size of mold, number of layers (often only one), and the choice of compression from one or both ends of specimen are variables in the static compaction test.

The vibratory method was developed mainly for compaction of granular soils because this method is one of the most effective means of achieving the maximum density in such materials.

Various vibratory techniques have been suggested and studied. The most commonly used method involves a table-type vibrator to which a sample container is affixed. Vibration is imposed by shaking the container either vertically or horizontally with sample under controlled surcharge. Based on several studies (Refs. 168 and 169, for example), a procedure using a vertically vibrating table method has been adopted as an ASTM Standard (ASTM D-2049). Yet, the method has been found to have many limitations (Ref. 170).

Another approach to vibratory compaction applies a vibrating tamper to the surface of the sample. This method has been widely used in Europe, and is very effective in achieving a relatively high density (Ref. 171).

The limitations of various vibratory compaction methods are 1) they do not recognize the difference between effects of impact of a weight and motion of the material particles, 2) it is hard to determine the magnitude of the compactive energy, and 3) conditions of vibration giving maximum density vary with the type of material and other compacting conditions as well.

The parameters needed to define the vibratory compaction testing conditions are the vibration frequency, amplitude, duration, surcharge, and specimen conditions. Further aspects of vibratory compaction will be discussed later in this chapter.

Gyratory compaction (Ref. 172) represents the latest development in laboratory techniques that attempts to duplicate the effect of field methods. The method simulates action of field compactors by providing gyratory shear while the soil is under a prescribed pressure. This is a useful method for understanding the basic compaction behavior of the soil, which simultaneously provides a measure of the build up of the strength and stiffness of the materials as it is being compacted.

Pressure level, number of cycles, magnitude of gyration angle, and mold size are among the factors defining the gyratory compaction test conditions. The equipment is very expensive, and not suitable for the large sample sizes required with ballast.

The unit weight of aggregate test (ASTM C29 and AASHTO T19) is also a type of compaction test, even though the purpose is slightly different from the above described tests. The test consists basically of rodding or tamping the aggregate sample into a mold in three layers, applying 25 strokes per layer with a 5/8-in.-diameter and approximately 24-in.-long steel rod. The method does not provide a high degree of compaction.

The test results are used for the concrete design purposes and for determination of quality of slag and lightweight aggregate, instead of for specifying the degree of compaction. Factors influencing the results of this test are given in Ref. 173.

Further research on laboratory methods suitable for ballast compaction was performed in this study to develop a simple and effective means of obtaining a reference density. The apparatus and procedures are described in Ref. 23. The approach uses a falling rubber-tipped weight to compact ballast placed in layers in a prescribed mold.

Different compaction methods have been discussed, and various parameters involved in the compaction processes have been indicated for each type of test. Other factors that affect results of the compaction tests are size and shape of mold, support of mold, sample preparation technique, type, magnitude and distribution of compactive effort, soil temperature, method of moisture determination, method of volume determination, layer thickness and total depth, degree of degradation during compaction, and soil type. Detailed discussions of each factor can be found in References 164 and 174. Results relevant to ballast are presented in Ref. 23.

5.2 VIBRATORY DENSIFICATION

As illustrated previously, cohesionless free-draining soils do not exhibit well-defined moisture-density relationships. Instead, relative density plays the major role in defining the state of compaction and the behavior of compacted soils. The calculation of relative density requires determination of maximum and minimum densities. The vibratory method is most effective in achieving maximum density of such soils. Considering the relevance of the subject to the ballast materials, vibratory densification behavior of cohesionless soils will be discussed in detail in this section. Methods of vibratory compaction in the laboratory have already been described in the preceding section of this chapter.

Influencing Factors. There are various factors controlling vibratory compaction behavior of cohesionless soils, and considerable effort has been devoted to delineating the effects of each of the controlling parameters and the mechanisms associated with vibratory compaction. The important parameters may be conveniently categorized as describing the 1) vibration, 2) surcharge, 3) test conditions, and 4) soil. Since the maximum density achieved from vibratory compaction is the result of the interaction among these parameters, it is difficult to separate one parameter from others and assess its effects on densification. In fact, the large number of parameters and their complicated interaction have resulted in various misleading and contradictory conclusions about vibratory compaction.

The vibration parameters include frequency, displacement, velocity, and acceleration. These are interrelated so that any two of them describe any vibration condition. Despite the interrelationship, acceleration has been identified as the primary vibration factor. The relationships obtained between density and acceleration, however, differ considerably, as shown in Fig. 5.2. The range of acceleration at which maximum densification occurs is 0.5g to over 2g, or anywhere in the tested range. Furthermore density may or may not decrease beyond the maximum, depending upon the type and amount of surcharge, mode and direction of vibration, and other test conditions. General trends that most investigators seem to agree with are that a surcharge reduces the densifying effect of vibration especially at high frequencies, and that over-vibration may occur, mainly at low frequencies and with no surcharge. Furthermore, most investigators agree on a vibration period of 1 to 5 minutes to reach

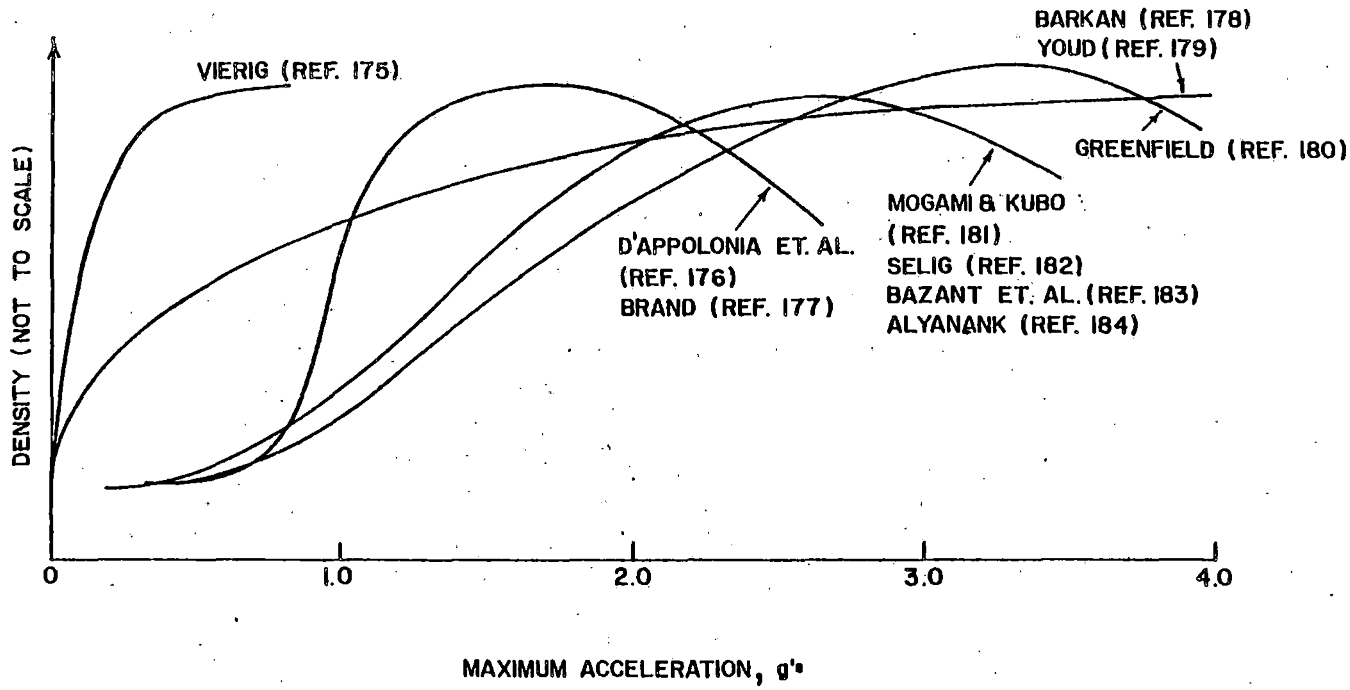


Figure 5.2. Densification of Granular Soils during Vibration as a Function of Acceleration, as Reported by Various Researchers

most of the densification.

Fig. 5.3 illustrates the interrelationship of the effects of the vibration parameters on the density. A series of density contour lines are shown as a function of frequency and acceleration, clearly indicating that for a given frequency or acceleration, density may either decrease or increase with an increase in either parameter.

The effects of surcharge on vibratory densification is illustrated in Fig. 5.4. This particular example was obtained from a test using air pressure to confine the sample during vibration. It is seen that as the confining pressure increases the acceleration required to initiate densification increases. This acceleration is believed to be the critical value which is required to overcome the contact forces between the particles. Little densification would occur below the level, and above it, a terminal density would be achieved dependent on confining pressure, but independent of the initial relative density.

The above described trends regarding to the surcharge, i.e., a density decrease with increasing surcharge, could well be reversed when a dead weight surcharge is used, depending on its magnitude and interaction with other influencing factors (Refs. 169, 171, and 186). An example is shown in Fig. 5.5, which illustrates that dead weight surcharge equivalent to 21.4 psi on the top of a silt compacted on a vibrating table improved densification compared to open mold tests without surcharge.

There are many factors associated with different testing conditions and techniques which affect the degree of vibratory densification of granular soils. They include mold size, moisture content, degree of saturation and drainage conditions during the tests. It is almost impossible to separate the effects

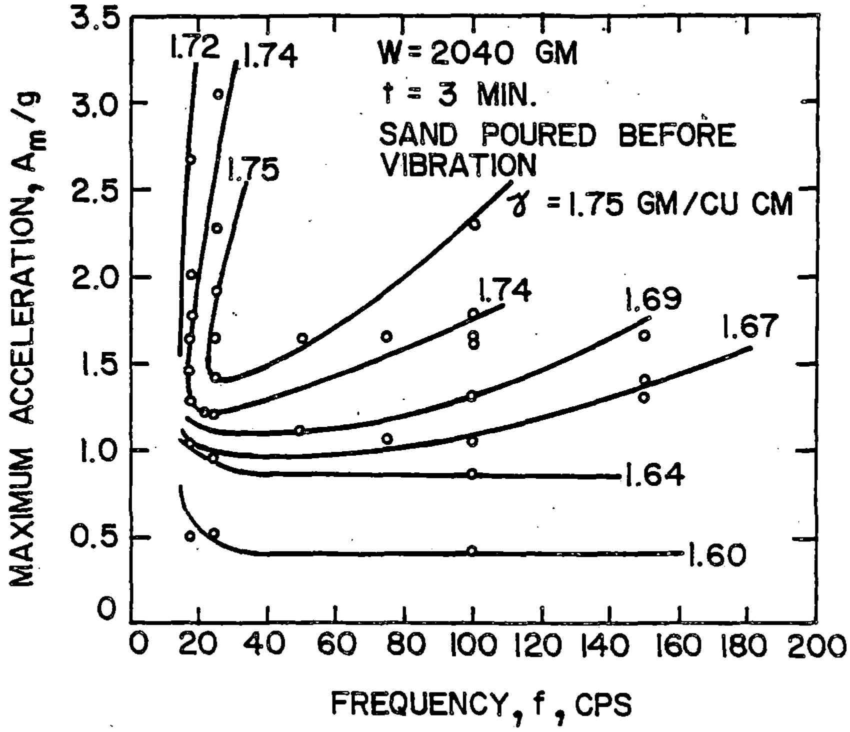


Figure 5.3. Contours of Constant Density as a Function of Frequency and Maximum Acceleration (Ref. 182)

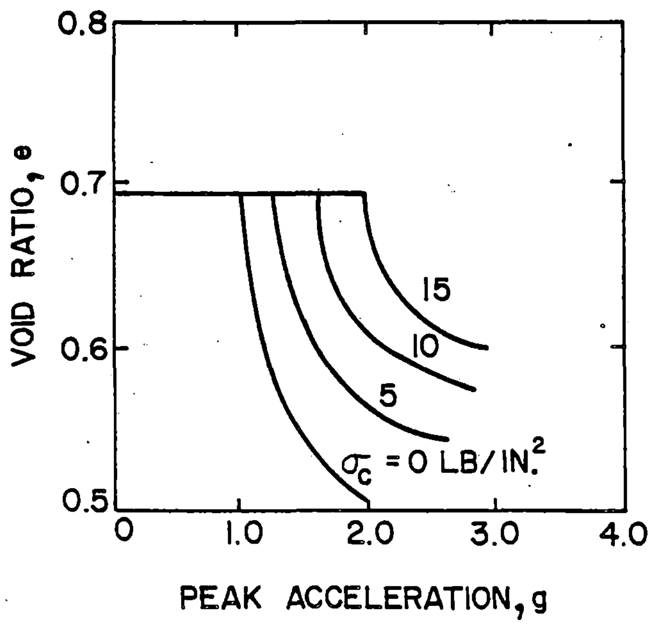


Figure 5.4. Densification of Ottawa Sand under Acceleration with Constant Stresses (Ref. 185)

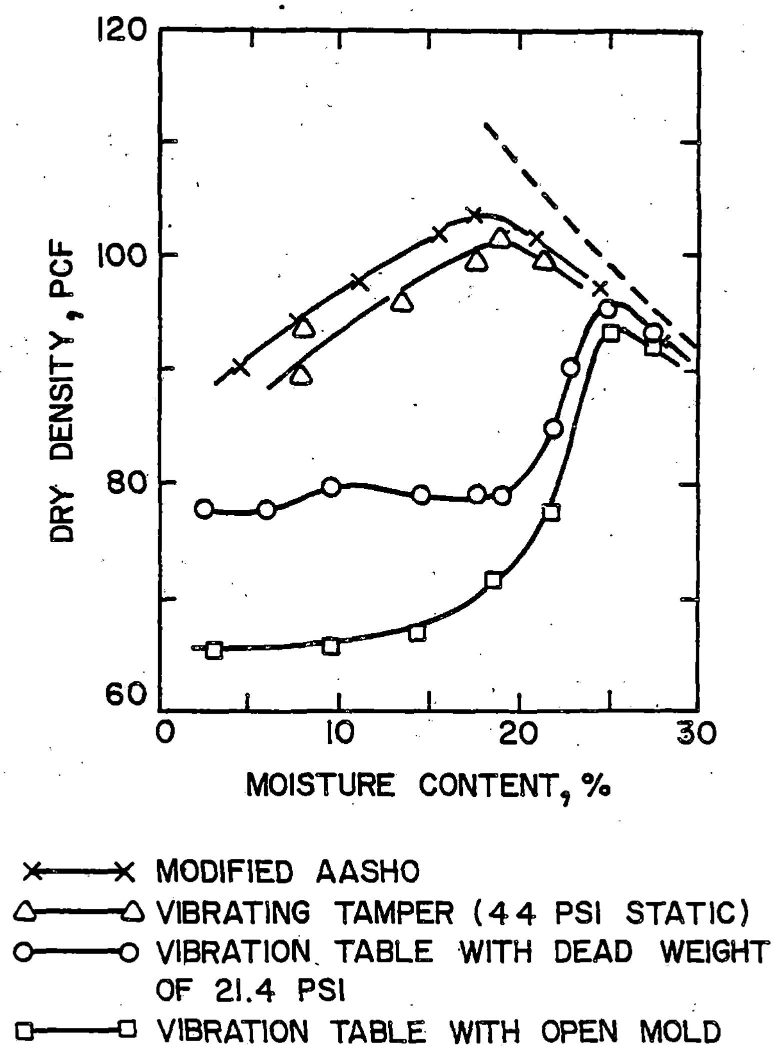


Figure 5.5. Density-Moisture Content Variation of Silt for Various Methods (Ref. 171)

of these parameters and generalize the trends. However, there are minor factors which would not change the basic principles of vibratory compaction.

The soil parameters include soil type, particle shape and size, gradation, specific gravity, and water content. Effects of these parameters during vibratory densification should follow the basic principles applied for general behavior of granular soils. For example, the soil with higher uniformity coefficient would compact more densely than those of lower values, if other characteristics remain the same.

Results with Gravel-Size Soils. The past studies of vibratory compaction of granular soils have been mostly for silt to sand size soils. Interest in vibratory compaction of particle sizes larger than sands seem to have originated for compaction control in rockfill dam construction (Refs. 187, 188, 189). According to Frost (Ref. 188), various methods of vibratory compaction ranging from vibrating tampers to vibrating table with 0.75 cu yd mold have been used to determine the maximum density of rockfill materials for various dam construction works.

No differences in the basic phenomena have been reported between the large size aggregates and sands. However, many difficulties have arisen with the former material. Large particle size requires a large size mold or container, and, therefore, heavy vibratory equipment is needed. In fact, the maximum densities of various rockfill materials determined with vibratory methods have been found to be lower than with other methods such as impact compaction (Ref. 187). Either the compactive effort was not enough or the optimum conditions of vibratory compaction were not achieved in those tests. Segregation of the sample during vibration also is a problem associated with vibratory compaction of large size soils, in addition to the difficulty in determining the density value.

The use of vibratory techniques in ballast tamping and compaction in the railroad industry have been the basis for some studies on vibratory compaction behavior of ballast materials, such as reported in Refs. 160, 161, and 190 through 193. The test results on various types of ballast materials ranging from crushed rock to slag, with particle size between No. 4 and 1-1/2 in. sieves, have shown the same general trends for density relationship to acceleration reported for sand-size materials. In general, maximum density was achieved with acceleration levels at 2.5 to 3 g's within approximately 20-30 sec of vibration time. However, a surcharge of up to about 5 psi increased maximum peak density compared to zero surcharge, and further increase in the surcharge reduced the amount of compaction at any acceleration.

<u>Mechanisms of Vibratory Compaction</u>. Four possible mechanisms for explaining the effect of vibration for compacting granular soils may be deduced based on the past studies. They are 1) particle vibration, 2) impact, 3) strength reduction, and 4) cyclic loading (Ref. 194).

The application of vibration to the soil causes individual soil particles to vibrate. As they vibrate, the particles can settle into either a more compact or a looser state, depending on conditions. However, a very small amount of cohesion between particles, even as little as provided by capillary moisture films in clean sands, can prevent this arrangement. Thus, particle vibration is believed to be important for dry or submerged granular materials and for clean ballast, but not for other materials or conditions.

The second mechanism speculates that the impact forces from the surcharge weight or compacting device during vibration is the major cause of compaction. This mechanism requires that the weight or device break contact with the material during the cycles of vibration, a situation that often does not occur.

The third mechanism is based on the fact that application of vibration can reduce the strength of the material and hence makes the material easier to compact. This mechanism is likely to be effective for ballast. However very little direct evidence is available to demonstrate the role of this mechanism.

The fourth mechanism is particle rearrangement from cyclic straining of the material produced by the application of vibration. This phenomenon has been clearly demonstrated (Ref. 194). Even without any significant acceleration or impact, cyclic loading has been found consistently to cause compaction. This mechanism seems to be the best explanation of why vibration works in any materials with cohesion.

5.3 FIELD COMPACTION

Equipment. A large variety of mechanical equipment is available for compaction of soils; however, they can be grouped according to basic types. Although overlap is likely with any subdivision, the following seems to be generally acceptable:

1) pneumatic tire rollers
2) vibratory smooth-wheel rollers
3) plate vibrators
4) smooth wheel power rollers
5) tamping rollers
6) segmented pad and grid rollers
7) rammers or tampers
8) track-type tractors.

Selection of compactor depends on various factors influencing compaction results in the field. Soil type and moisture condition often dictate type of equipment and methods of use. The intended function of the compacted material is one of the important factors to be considered. Table 5.1 summarizes characteristics of various compactors. Detailed discussions on the subject should be referred to Refs. 196, 197.

Among the above-mentioned compactors, vibratory compactors have been recognized as the most effective means of compacting granular materials as well as rockfill materials. Especially in ballast compaction, methods using vibratory tamping or plate vibrators have become a major technique.

The factors influencing field compaction results include soil type, water or chemical additives, methods of compaction, method of preparation, uniformity of procedures, and the environmental parameters. Generally, the effects of these factors in the field would be the same as in the laboratory, although in the field variability of conditions often makes it difficult to detect the individual effects of each parameter. Proper understanding of the effects would help engineers in the field to select the right compactor for a given job, and the appropriate conditions of moisture, layer thickness, and number of coverages or compaction time.

In the past, little progress has been made in the formulation of equations which explain and predict compactor performance. Even determining the principal parameters for each type is not complete; and, in fact, is still a subject of much disagreement. There is also an uncertainty about the influence of some of the parameters, such as the role of inflation pressure for pneumatic rollers, and frequency for vibratory rollers.

A set of equations for defining compactive effort of basic classes of roller has been developed (Ref. 198). Expressions are derived for calculating production rate and horsepower requirements, based on the concept of work done

Table 5.1. Compaction Equipment and Methods

Type	Characteristic Variables (Both dependent & independent)	Applicability (Ref. 196)
Smooth-wheel self-propelled	Number of rolls, gross weight, roll dia., roll width, lb/in. of width	Appropriate for subgrade or base course compaction of well-graded sand-gravel mixtures. May be used for fine-grained soils other than in earth dams. Not suitable for clean well-graded sands or silty uniform sands.
Pneumatic-tired rollers	Number of wheels, gross weight, wheel spacing, type of roller, inflation pressure, contact area, tire size.	For clean, course-grained soils with 4 to 8 percent passing the No. 200 sieve. For fine-grained soils or well-graded, dirty coarse-grained soils with more than 8 percent passing the No. 200 sieve.
Tamping roller (sheepsfoot, segmented)	Gross weight, type of foot, number of feet/drum, area of foot, contact area percent of total area, length of feet, width of drum, contact pressure	For fine-grained soils or dirty coarse-grained soils with more than 20 percent passing the No. 200 sieve. Not suitable for clean coarse-grained soils. Particularly appropriate for compaction of impervious zone for earth dam or linings where bonding of lifts is important.
Vibratory roller	Gross weight, drum weight, roller width and dia., frequency, amplitude, centrifugal force, speed of travel.	For coarse-grained soils with less than about 12 percent passing No. 200 sieve. Best suited for materials with 4 to 8 percent passing No. 200, placed thoroughly wet.
Plate vibrator	Gross weight, weight of vibrating plate, contact area of plates, contact width, frequency, amplitude, centrifugal force, speed of travel.	
Tampers	Weight, area of base, height of jump, frequency.	For difficult access, trench backfill. Suitable for all inorganic soils.
Track-type tractors	Gross weight, width of track, contact pressure, speed of travel.	Best suited for coarse-grained soils with less than 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.

by the towing unit through drawbar force. Although the compactor models used are elementary, they do provide a first approximation for these three system characteristics (compactive effort, production rate and horsepower) in terms of the basic controlling parameters, including compactor weight, rolling width, layer thickness and travel speed. The resulting expressions have been checked using available data on compactor performance. Unfortunately, a direct measure of compactive effort has almost never been made in all of the field testing conducted in the past which is reported in the literature. This and other basic omissions make it impossible to obtain an accurate check on the equations and a precise estimate of the coefficients required. Nevertheless, the approach taken is suitable for a preliminary compactor analysis, and it is believed to be the best approach available at present for analyzing compactor performance to aid in developing design specifications and in selecting compactors for a given job. Further work on methods of compactor analysis is needed.

<u>Compaction Control</u>. The purpose of compaction is to improve one or more engineering properties of a given material, and hence, to improve the performance of the material in a given application. Therefore, conceptually any material properties related to the performance improvement sought during compaction could be used as criteria indicating the effectiveness of compaction and determining the level of compaction required in a given application. These include strength, compressibility, resiliency, wave velocity, and density. The problem then is to determine what magnitude of these properties are desired and how to specify and control them. By far the most widely used property for control is density.

Density is the most widely and frequently used property for the compaction measurement, even though it may not be the best. The popularity seems to originate from the facts that density along with moisture content can in general define the compaction behavior of soil reasonably well, and limiting values of density can be determined with relative ease for most materials. Previously used field methods for measuring the density of compacted materials are described in Ref. 23. However, ballast is quite different from most highway materials. None of these methods are suitable for measuring ballast compaction. Thus, further research was conducted to develop a method. The results of this effort are presented in Ref. 23.

A method which measures the velocity of seismic waves traveling through the material mass has been used to determine the compactness of the material. The seismic wave velocity is related to both the mass density and the elastic modulus of the material. The velocity of the wave generated by the impulse of a hammer striking a metallic plate or spike on the ground is measured with a seismograph. This is done by determining the elapsed time for the wave to travel from the triggering device to the sensing device.

This non-destructive method can cover a large area of a compacted zone in a short time period, without any material preparation. However, attempts to correlate the seismic velocity with density and other soil properties have so far been relatively unsuccessful, even though it has been clearly demonstrated that velocity is quite sensitive to changes in compaction (Refs. 135, 142).

Measurements of strength and stiffness have often been made on compacted soils to obtain design information and determine the degree of compaction as well. However, only the plate load tests can be used on ballast. The plate bearing tests indirectly measure the degree of compaction of the soil from the vertical force-deflection characteristics under a given size of bearing area. This method, which is relatively rapid and non-destructive, may be used on a variety of materials. A review of this method is given in Ref. 23. Further development of the plate load test for ballast use is also described.

An alternative method of compaction measurement has recently been suggested, using an inductance type gauge (Ref. 199) which measures the soil strain by the change in the electro-magnetic coupling between pairs of disc-shaped sensors embedded in the material being compacted. This relatively new approach has been evaluated and shown to be successful in granular materials in a series of field compaction tests (Ref. 200). One of the major advantages of the method is a possibility of continuous monitoring of the material behavior during and after compaction. It is particularly useful in case of coarse-grained materials in which other conventional density measuring techniques are not suitable.

Compaction Specifications There are two basic approaches for specifying compaction. The first, and oldest, is "method specification," in which factors such as type of compactor, speed, layer thickness, and number of passes or compaction time are prescribed. The second is "end result" specification, in which minimum properties required of the final product are stipulated. Presently, a minimum density is by far the most commonly used performance criterion.

The trend in highway construction in the United States has been increasingly in favor of the end-result approach, because it is more flexible and gives the contractor the opportunity to select the construction method most suitable to him. The obvious limitation of this approach is its dependence on the quality of criteria used and the reliability of the method for determining whether the objectives have been achieved. While this approach may be simple in concept, it is difficult to achieve in practice. The reasons for this difficulty include the frequently large point-to-point variability of the compacted state of the material coupled with the inability of present measurement techniques to sample a sufficient number of points rapidly enough to keep pace with the construction. In practice, then, a combination of the "method" and the "end result" approaches is usually employed.

5.4 BALLAST COMPACTION CONSIDERATIONS

The ballast materials being presently used are characterized as open-graded, coarse-grained aggregates with free drainage. They are also initially clean. Such materials pose difficulties in determining and controlling the amount and state of compaction.

Typically, such materials do not exhibit the well-defined maximum dry density at an optimum moisture content. The resulting density for a certain compactive effort is sensitive to changes in gradation, degradation, and moisture content. Furthermore, reference density and optimum compaction conditions are hard to establish. There is also a lack of compaction testing methods for this type of material. The standard procedures used for highway and airfield construction are not adequate.

The above problem becomes compounded in the case of field measurements. During mechanical compaction the compactive effort is usually concentrated in certain areas, such as below the ties and cribs, near the rails, and shoulders. Therefore, the resulting compaction is nonuniformly distributed over the whole area of ballast. The traffic induced compaction is also believed to occur in the limited area under the ties. The nature of non-uniformly distributed compaction may require separate specifications for ballast compaction in different areas, and will require measurement methods that can properly test the areas affected by compaction.

There are two basic approaches for the ballast compaction measurements.

One is to measure the compaction itself, and the other is to determine the effects of ballast compaction. The former approach includes density tests, strength or stiffness tests, or the evaluation of the ballast structure change before and after compaction. The latter approach is mainly concerned with the track geometric configuration and track stability changes due to ballast compaction, such as track settlement, lateral and longitudinal tie resistance, track modulus, and track geometry. The problem with this approach is the difficulty in separating out effects of compaction from the effects of other factors like traffic or subgrade conditions or environmental.

The most frequently method of density measurement for ballast has been the nuclear method (Refs. 201, 202, 203, and 204). This method has been widely tried in European countries, and various versions of instruments devised. In any case, this method requires placing a probe(s) into the ballast, which causes a change of ballast conditions, especially in the neighborhood of the tube.

Various replacement methods have also been used in ballast density measurement. According to Riessberger (Ref. 202) and Birman (Ref. 203) water, sand, or gypsum replacement methods have been tried to a limited extent. However, the quantity of samples required was so large that these methods were not suitable for use in track structures.

Plate load tests have also been used for ballast compaction evaluation (Ref. 202), by measuring the force-deformation characteristics of a plate on the ballast. A detailed procedure has been suggested by CNR (Ref. 137).

Track settlement surveys, in which both differential and overall track settlement are determined to indicate the effects of ballast compaction, may be used as an indirect measurement of ballast compaction (Ref. 205). However, the evaluation of settlement is not a suitable means to measure the effect of ballast compaction, because it depends on many other parameters of greater influence.

Tie displacement tests determine the effect of ballast compaction on the resistance of ties to displacements in either lateral or longitudinal directions. There are two different methods of lateral tie displacement tests: single-tie test, and multi-tie test. Details of the procedure involved in each of the methods may be found in Ref. 205. The lateral resistance is so much dependent on the lift during tamping that the test results may not indicate the specific effect of ballast compaction, especially when the measurements are made immediately after tamping.

The tie load test, or track modulus test provides a measure of vertical track stiffness which could be used to compare compacted and uncompacted areas. It has been long used to obtain design information for the track system. Details of the tests were discussed in Section 2.

After considering the various alternative approaches for measuring ballast compaction, three methods were selected for further development. These are, 1) water replacement for density determination, 2) small plate load test for vertical ballast stiffness determination, and 3) resistance of a single unloaded tie to lateral displacement. This work, including the test procedures and the type of results obtained, is described in detail in Ref. 23.

The objectives of ballast compaction are 1) to limit, if possible, the settlement of the track under the effect of traffic loadings, 2) to avoid differential settlement as the track deflects under load, and 3) to assure a resistance, immediately after maintenance work, to lateral and longitudinal track displacements which will guarantee track stability through the restraint deformation as a consequence of traffic and temperature variations.

The following must be taken into account to determine the optimum compaction level for given compaction conditions:

1. the "anvil" effect of the foundation

2. ballast particle size and the overall dimensions of the ballast section as well as the gradation of the ballast material, and

3. existing conditions prior to compaction, such as the quality of tamping and train traffic from the completion of tamping up to commencement of compaction.

The time interval for compacting ballast as well as appropriate conditions of compactor operation should be determined based on a careful consideration of various factors including track conditions, the level of performance, and economic factors.

The probable optimum level of ballast compaction is dependent on the ballast properties, ballast conditions after tamping, and the required level of performance. At present no information is available to establish suitable quantitative ballast compaction specifications.

6. PRESENT PRACTICE OF BALLAST RELATED TRACK CONSTRUCTION AND MAINTENANCE

Track construction can be interpreted as the entire sequence of operations associated with either 1) the installation of new track, or 2) as a major rehabilitation (reconstruction) of existing track, including rail and tie removal, and the addition of extra ballast material for the track bed.

The current situation of the railroad industry in the U.S. indicates that new track construction for distances greater than a few miles is uncommon and probably not economically justifiable when compared to the costs involved with upgrading the existing, and possibly poor quality, lines. Furthermore, track construction procedures, specifications and particularly costs are highly dependent upon the locality, which prohibits generalizations to be made for new track construction. General construction procedures for new track are specified in AREA Manual (Ref. 2). Detailed discussions of new track construction will not be presented in this report since the amount of new track construction is extremely small compared to the amount of upgrading of existing lines. Thus, the emphasis in this report will be placed upon the rehabilitation of existing lines with on-track equipment.

This research effort concerns the physical state of the ballast material in the track bed and the relationship of this state to track performance. Understanding the maintenance processes which affect the state of the ballast material is necessary. Thus this chapter will emphasize ballast-related maintenance operations.

The main categories of track maintenance work which will be discussed are 1) track renewal, 2) tie and rail replacement, 3) ballast cleaning, 4) ballast hauling and distribution, 5) ballast shaping, 6) track surfacing and lining with ballast tamping, and 7) ballast compaction. A complete compilation of information for all types of existing track maintenance equipment in each of these categories was not feasible. However, an adequate coverage of the subject was obtained from readily available literature. Equipment types and procedures used for these maintenance operations may differ between railroads, since the experience of the maintenance-of-way department and the available materials, labor and equipment will influence the approach taken.

6.1 TRACK EVALUATION

Maintenance of a given section of the track structure is required if one of the following conditions indicates the need for an improvement of track conditions: poor riding quality, excessive lading damage, derailments, visual inspection indicating deterioration of the track components, or exceeding the track safety specifications which are recorded from track measuring cars. Hay (Ref. 206) states that although track geometry and track material can be discussed separately, they do not act independently. Other things being equal, the material in a track with good geometrical properties will outlast that in a track with poor line or surface. Conversely, sound track material will help preserve geometrical quality while material weaknesses act to destroy geometrical quality so that permanent deformation is caused by loads which could otherwise be easily carried. Poor geometry also hastens material wear by increasing the magnitude of dynamic loads imposed on the track and its elements.

The track measuring car method is currently used by some railroads and provides a more efficient, reliable, and quantitative means than visual observation for pinpointing already hazardous or developing track problem areas over many miles of track. The most efficient distribution of track maintenance equipment, manpower and material is essential. With track recording cars reliance does not solely have to be placed on subjective opinion as to the quality of track by different track supervisors on different stretches of track. However,the track geometry car should not be considered a panacea for track maintenance problems, but should still be supplemented by visual track inspection, since determination of the many components comprising the track system can occur and not necessarily be identified by the track geometry car.

Examples of the manufactured equipment which records the deterioration of the track geometry are given in Table 6.1. However, some railways have designed and built their own such equipment. Track geometry, inspection, recording, measuring, and evaluation are the designations given to these vehicles. However, each performs one or more of the same basic functions of measuring the variation in gauge, surface, line and twist of the track. Differences in measurement equipment involve type and spacing of the measuring wheels and methods of measurement, measuring devices, and computer facilities for the data obtained.

At the FRA track program review (Ref. 210), Meacham of Battelle-Columbus Laboratories and Corbin of Ensco, Inc., presented research progress reports on their development of track measuring systems. While not serving the same function nor providing the same capabilities of manufactured track geometry defects, their efforts are concentrated towards a means of evaluating track geometry defects. Meacham is developing an instrument to measure track impedance which operates from an intermittently stationary car, while Corbin is developing a continuously moving measurement of vertical track stiffness using the Southern Railway's track measurement car.

Table 6.1. Track Recording Car Descriptions Based on Manufacturers' Literature

Make	Model	Type	Total Weight (kips)	Measurements
1. a) MATISA (Ref. 207)	M422	Recording	50.6	7 Parameters: longitudinal level, 3 for twist, superelevation, gauge, versines
b) MATISA	WE200 (with B200 series tamper, Table 6.5)	Recording	----	4 Parameters: superelevation, twist, level and alignment
2. a) TAMPER (Ref. 207)	----	Valuator	----	7 Parameters: gauge, twist, superelevation, left & right surface, left & right line
3. a) PLASSER & THEURER (Ref. 208)	EM80	Measuring, Evaluation	70	12 Parameters: Lateral acceleration of car body, gauge, vertical irregularities in the surface of both rails, superelevation, speed, mile post marker, special events marker, curvature, twist (or crosslevel), profile and alignment of both rails
b) PLASSER & THEURER	EM50	Measuring, Evaluation	----	1st track car
c) PLASSER & THEURER	EM100	"	70	Same as EM80
d) "	EM40	"	----	Not for high measuring speeds
e) "	Track Testing Wagon	"		
4. a) TRANSMARK (Ref. 209)		Track Geometry	----	Parameters: top left & right, alignment, twist, crosslevel (normal), cant & dynamic, gauge, horizontal and vertical curvature, vertical slope longwave, equilibrium speed, vertical and lateral ride index, vertical slope left and right, horizontal slope, vertical and lateral bogie acceleration, distance travelled

Plasser literature (Ref. 208) illustrates the "fault" zones which are most likely to lead to changes of track geometry and discusses the measurement of a particular track parameter. It also indicates the following effects that a change in a track parameter has on train traffic:

1. Gauge: Variations can lead to uneven car and rail wear.

2. Surface Irregularities: Improper seaming, missing ties, or washed out roadbeds lead to ride discomfort for passengers and damaged cargo.

3. Rail Profile: Changes in longitudinal level lead to vibration and bounce difficulties, especially at high speeds.

4. Rail Alignment: Horizontal irregularities in the rail damage both freight and car.

5. Superelevation or Cross level: Improper superelevation can lead to excessive and uneven wear at wheel-rail contact points and to derailments of trains negotiating those curves at high speeds.

6. Twist Measurement: Metal fatigue and roadbed erosion cause distortion of two rails which will cease to run at parallel levels. The car axles become twisted causing increased car and track wear, and cargo damage.

Blanchard (Ref. 211) also states that gauge, line and level are all interdependent. He states that gauge is the most critical and is most likely to influence a change in line and level.

An important advantage of track recording cars is that a means is provided to quantitatively compare track conditions over a railway system with track standards which have been set. Other advantages as reported by the equipment manufacturers are as follows:

1. A means is provided for railway administrations to plan trackwork management and maintenance policies.

2. Recording is accomplished under dynamic load similar to that of normal trains.

3. No speed reduction is needed when passing through switches.

4. Parameters measured can be graphically plotted as well as stored on tape and analyzed on a computer.

5. For each parameter measured, indication of the amplitude of faults above the designated safety level, and the exact location is provided on the records by the computer.

6. The recording cars can physically mark faults on the track using a paint marking device (Ref. 209).

7. The parameters measured may provide a good tool to indicate the effectiveness of ballast compaction (Ref. 212).

6.2 TRACK RENEWAL

The largest scale track maintenance operation is the removal and replacement of ties, rails and fastenings of the track superstructure in a single continuous process rather than replacement of the individual track components. The track renewal train or track relaying train with this assembly line principle was developed specifically with this concept in mind. Burns (Ref. 213) states that the prime advantage of this method of maintenance is a significant reduction in track maintenance costs and track time requirements. However, such equipment, briefly described in Table 6.2, has not been used in the past U.S. track maintenance programs.

The widespread adoption in Europe of concrete ties and continuous welded rail has created the need for machines capable of replacing ties out of face. European maintenance of way machinery manufacturers have built machines for this purpose using essentially four different approaches. These machines

Table 6.2. Track Renewal Trains

Reference	Manufacturer	Model	Production	Comments
1. (a) 207	Matisa/ Canron	P-811 Valditerra System	660 yd/hr (rated) Min. ave. 10 ties/min Max. ave. 17 ties/min 1640 ft/hr (Italy) [825 ft/hr Burns (Ref. 213)]	Tangents and curves as small as 3-1/2°; contains roadbed plow and ballast compactor behind plow; controlled tie spacing (automatic); can change ties or rails, exclusively; used in tunnels, bridges, stations
2. (a) 208	Plasser & Theurer	SUZ 350	1150 ft/hr [900 ft/hr Burns (Ref. 213)]	Tangents and curves, 2 'machines,' 3 with ballast cleaner
(b) "	"	SUZ 2000I	820 ft/hr	Gantry crane, old track removed in panels and new track laid tie by tie
(c) "	"	SUZ 500	[825 ft/hr Burns (Ref. 213)]	One 'machine'; can also remove old track in panels
(d) "	"	SUZ 500B		One machine removing and laying track with long welded rails and individual ties; is specially suited for working in short track possessions.
(e) "	"	SUZ 500LS		One machine similar to above except one end of machine on crawler. Can build new track.
(f) "	"	SUZ 500j		One machine removes the old track in panels and lays the new track tie by the tie and welded rail
3. (a) 214	Secmafer	216 M8/M9 401 BR 48-61/63		Gantry crane, old track removed in panels and new laid tie by tie
4. (a) 215	Modern Track Machinery			Gantry crane, old track removed in panels and new laid tie by tie
5. (a) 213, 216	Railways of U.S.S.R.			Gantry crane, old track removed in panels and new track installed in panels. Rail later changed for C.W.R.

have also been used for replacing wood ties with wood ties. Brief descriptions by Burns (Ref. 213) of each of the four basic type of machines follow.

The Valditerra System, developed by Matisa, removes the rail and old ties and installs new ties and rail within one machine and within a short distance. Plasser and Theurer have also developed a similar machine. The long-term average production rate of this type of machine is approximately 825 ft/hr with a 1/2 hr start up and 1/2 hr clean up.

The two-train system, an earlier Plasser and Theurer development, has two self-contained machines working relatively independently of one another. The first picks up the track and removes the ties and rail. The second installs the new ties and rail. Between the two machines is a length of track that is clear of both ties and rail. In this section, it is possible to perform such operations as subgrade stabilization and ballast cleaning by off-line equipment. The continuous production rate of this system is approximately 900 ft/hr with a 3/4 hr start up and 3/4 hr clean up period.

With the panel-track system, developed and used extensively in Russia, panels of the old track are removed by a specifically designed on-track crane, moving backward and pushing flat cars ahead of itself. Panels of new track are laid by a similar crane pulling flat cars of new panels behind it. The jointed rail is later replaced by continuous welded rail.

A system, developed by the French, uses a gantry crane which, while running on the new continuous welded rail that has been previously laid on the shoulder, picks up panels of the old track, and also places the new ties. Another machine positions the new rail on the ties.

In principle, the Matisa P811 or the recently developed Plasser and Theurer SUZ 500, are similar in operation (Ref. 213): A single machine removes and installs the new track within a short distance, thus precluding any operation on the ballast other than levelling off the cribs. These machines receive their alignment from the original track, but the super-elevation can be changed. The Matisa P811 also incorporates vibratory plates to compact the level ballast bed after tie removal and plowing of the crib material. The SUZ 500 may or may not contain this feature. The average production rate of this type of machine is 500 ties per hour or 825 feet per hour, with 1/2 hour for start-up and 1/2 hour for clean up (Ref. 213). However, the rated production rate is 2000 ft/hr for the Matissa P811.

6.2 RAILS AND TIES

Decay, abrasion, fatigue and breakage of wood ties due to train loading and environmental factors periodically necessitate the replacement of this component. The average lifespan of wood ties on mainline track is estimated to be 25 years. Some U.S. railroads are currently considering introducing concrete ties as an alternative to wood ties with the expectation of increased levels of track performance. Foreign railroads have experienced satisfactory results with concrete ties. However, with the higher axle loads in the U.S., the effect on the track maintenance cycle is still unknown. Failures of concrete ties occur with fracture at the rail seat area or cracking of the tie.

Since wood ties are far more common, only the procedures for their replacement will be discussed. The main steps observed in one field operation are as follows:

1. A mechanical spike puller is used to remove the spikes from the ties marked for replacement. The tie plates are also removed.

2. A tie cutter is used next to cut the ties into thirds. The center third of the cut tie is picked up by the machine and deposited on top of the crib, while the outer thirds are mechanically pushed outward onto the shoulder. The pieces may be chipped, buried or carried away.

3. A ballast scarifier with rotating blades cleans out the ballast where the old tie was embedded so that the new tie can be inserted easily. Ballast collected under the rails during the scarifying operation is leveled by the same machine.

4. A tie inserter places new ties part of the way under one rail. Then the tie is mechanically pushed into its proper position on the ballast bed.

5. The tie plates and spikes are replaced. If necessary tie spacing and gage are first adjusted.

A complete and illustrative manual of rail defects has been compiled by Sperry Rail Service (Ref. 217). Serious defects, such as excessive wear or breaks, would require replacement of the rail section. In the U.S., defective jointed rail is often replaced with continuous welded rail.

Procedures for handling, transporting, unloading, and installing rail are described in AREA Manual (Ref. 2). Few manufacturers' specifications on rail laying equipment were available. The new rail is loaded in the yard on rail handling cars by forklifts or cranes, and transported to the desired track section and unloaded on the shoulder by cranes. Rail fastenings are removed by conventional methods. The defective rail may be removed and the new rail installed by using derricks, cranes or a sleeper-positioning-and-rail-displacing machine, such as the Plasser Theurer IP 101. Rail fastenings are subsequently reconnected.

6.4 BALLAST CLEANING

Increasing costs for material, and for handling and transporting of new ballast have caused maintenance of way engineers to seek alternate methods by which to restore badly fouled track-beds with a clean, uniformly-graded ballast material. Crib and shoulder ballast undercutters and cleaners represent a solution. This equipment performs the functions of removing and screening the ballast existing under the track and then redepositing the desired gradation of material into the crib. Separated material coarser and finer than the selected gradation limits of the ballast redeposited in the crib is placed by conveyor at the side of the track or loaded into open cars. A selection of available ballast cleaning equipment is described in Table 6.3.

Plasser and Theurer, Canron, Secmafer and Kershaw produce the heavy-duty ballast undercutters and cleaners to perform the complete operation on tangent and curved track. The Plasser and Theurer model RM 74 U has the additional capability of undercutting and cleaning at switches. The Plasser machines are also capable of treating the subgrade with a bitumen spray, or introducing a sand blanket or membrane on the subgrade while performing undercutting and cleaning. The Secmafer machine (214) levels and compacts the ballast such that the track can be immediately returned to service. The rated productions are from 460 to 850 cu yd/hr, but these numbers are dependent upon machine working speed, depth of cut under the tie, degree of difficulty in disaggregating the ballast material and physical obstacles such as road crossings, bridges, and switches.

Smaller machines are available to remove only the crib, or the shoulder or crib and shoulder but do not clean the ballast. In some cases the shoulder

Table 6.3. Ballast Undercutters and Cleaners

Reference	Manufacturer	Model	Production	Comments
1. (a) 208	Plasser & Theurer	RM62	—	Can put barriers on subgrade, i.e., bitumen spray, sand blanket, styropore sheets (u & c)*
(b) "	"	RM63	850 cu yd/hr	Can also lift, line, profile (u & c)
(c) "	"	RM74U	720 cu yd/hr	Also used on switches and crossings. Can introduce sand blanket. Depth of cut 10 to 30 in. (u & c)
(d) "	Windhoff	FR312	—	Crib ballast remover (r)*
(e) "	Plasser & Theurer	UFR80	—	Shoulder cleaner (sr & c)* Is used with Windhoff FR312
(f) "	"	CR312	—	Crib cleaner; can be equipped with shoulder plow (r)
2. (a) 207	Matisa/Canron	C311	Up to 460 cu yd/hr	Depth of cut 4 in., minimum under tie, to 30 in., maximum from top of rail (u & c)
(b) "	"	C330 (Paganelli system)	785 cu yd/hr	Depth of cut 12 to 25.5 in. (u & c)
(c) "	Tamper	GO-4 Trac-Gopher	3 ft/min undercutting 8 ft/min trenching	5 in. maximum cutting depth, cut shoulder and under tie outside rail, used at road crossings, approaches to non-ballasted bridges, one side at a time (r)
3. (a) 218	Kershaw	72	600 cu yd/hr	4-1/2 in. min. cut to 12 in. max (u & c)
(b) "	"	2FU	—	— (u & c)
(c) "	"	42-1	—	Switch undercutter, 5 in. to 6 in. depth (u)*
4. (a) 219	Marmon	—	—	Switch undercutter, requires 2 passes (u)
5. (a) 220	Speno Ballast Cleaning Service	—	1 mile/hr	8 in. cut, requires 2 passes Plow shoulder, then undercut track (u & c)
6. (a) 221	Loram	—	—	Crib skeletonizer, bridge approaches and switches also, 3 in. depth, in crib only (r)
(b) "	"	—	1 mile/hr	Shoulder ballast cleaner, also shapes slope (sr & c)
7. 214	Secmafer	400MH-02	785 cu yd/hr	Levels, compacts ballast, aligns track to such degree track can be released for traffic immediately after cleaning (u & c)

*u & c = undercuts and cleans; u = undercuts; sr & c = shoulder remover and cleaner; r = removes ballast only material is cleaned and redeposited at the tie end.

Loram (Ref. 221) produces dual machines, called Autotrack and Autosled, which raise the track, plow out the fouled ballast bed to the shoulder area, or sled the ballast to distribute the crib material underneath the ties, and replace defective ties. Reballasting normally follows this operation. This method does not incorporate rail removal and replacement.

One ballast undercutter and cleaner observed in operation was model RM 74-UHR, manufactured by Plasser and Theurer Company. Total weight of the machine is 70 tons. It has a working speed of about 10 to 15 feet per minute. The undercutter is capable of cutting laterally up to a distance of two feet away from the end of the ties and to a depth of ten to 33 inches under the tie. One operator is required, along with a mechanic and one or two laborers. Near the center of the machine a large hydraulic ram clamps underneath the rail heads and lifts the track up about one inch above the original level. A large continuous chain with cutting teeth passes underneath the tie. These teeth scrape out the ballast from under the tie, carrying it to the shoulder, where it transports the ballast up into the machine, where a screen shaker is located. The cutting chain is not placed parallel to the bottom of the ties, but is pitched two inches downward on the outside rail for easier collection and transport of the ballast and to provide a sloped surface for drainage. The large shaker inside the machine contains various sized screens with square openings. Material whose particle size is either above or below the range of screen sizes is discarded in a windrow about 15 ft from the edge of the track by an overhead conveyor. The remaining ballast particles are redeposited in the crib in the previously undercut sections by another set of conveyors.

The amount of material removed in the cleaning operation represented as much as 1/3 to 1/2 of the initial ballast volume, but because this was mostly fines filling the voids of the larger stone particles, the bulk volume of the cleaned ballast was not reduced by as much volume as that of the material removed. However, additional required ballast was provided by a ballast train. The cleaning operation was not very effective during rain, because the fine material tended to cake when wet, thus clogging the screens. To what extent the material from the various screens was remixed before depositing under the track was not determined.

6.5 BALLAST DISTRIBUTION AND SHAPING

A ballast train is required to replenish the ballast material in the crib after a cleaning operation or deposit additional material in the crib (reballasting) before or after a major lift.

A ballast equilizer or regulator is needed to fill the cribs and to shape the shoulders of the ballast after raising and tamping. In operation, excessive ballast between the rails is plowed to regions outside of the rail, into the crib and also onto the shoulder. One or two passes may be required for the shaping and filling operation depending upon the amount of ballast material to be moved. The regulator can also place ballast in the crib for reballasting purposes by transferring ballast from the shoulder. This machine is equipped with a shoulder former to shape or dress the shoulder to the desired slope and shoulder width at the tie end. This operation can be performed simultaneously with material being plowed out from the track center. The final operation of removing excess ballast not plowed out from the track center as well as ballast which has accumulated near the rails is performed by the ballast broom or sweeper. Shoulder forming and sweeping operations can be accomplished by the ballast equilizer when moving in either direction. However, uniformity of geometric appearance and cleanliness of the track structure are also obtained prior to and after track tamping.

Other functions of a ballast regulator as reported by Kershaw (Ref. 218) are: 1) dressing ditch in multiple track, 2) scarifying and deweeding ballast shoulders, 3) breaking up mud pockets at tie ends, and 4) plowing ballast from tie ends to improve drainage.

A representative list of ballast regulators is given in Table 6.4 In addition to the function of ballast regulating, the Jackson spreaders and Plasser and Theurer profilers shape the profile of the embankment.

6.6 SURFACING, LINING, AND TAMPING

The track geometry consists of crosslevel (surface) and longitudinal alignment (line). Surfacing and lining operations are performed simultaneously or in close conjunction to correct track geometry defects. Applications include major out-of-face trackwork requiring reballasting with 4 to 6 in. raises, for skinlifting involving track raises up to 1/2 to 2 in., or spot work in short track sections.

Fundamentally, three processes are involved: 1) raising the track to the proper elevation, 2) adjusting the track horizontally to align it, and 3) tamping the ballast to repack it in the voids under the tie near the rails. Surfacing and lining operations require tamping to fill voids under the tie created by repositioning. However, tamping may be done alone by a machine that does not have controls for surfacing and lining if the track is already in proper position and it only needs ballast to be packed under the tie.

Ballast in the crib remains in a loose state after tamping thus reducing lateral tie resistance and lowering bearing support underneath the tie

Table 6.4.

	Reference		Make
1.	(a)	207	Tamper
2.	(a)	218	Kershaw
3.	(a)	208	Plasser & Theurer
	(b)	"	"
	(c)	"	"
4.	(a)	222	Jackson
	(b)	"	"
	(c)	"	"
5.	(a)	219	Marmon

Ballast Regulators or Equilizers

Model	Equipment
BEB-17	Equilizer
26-1	Regulator
PBR-103	Regulator
SSP80	Rapid ballast profiling
USP3000C	Universal ballast distributing and profiling
4-200	Straight wing spreader
4-150	Spreader/ditcher
4-100	Spreader/ditcher with high snowplow
Track Patrol	On/off dressing machine

compared to the undisturbed ballast state. Opinions differ about the state of the ballast beneath the tie after tamping.

The specifications for Canron, Plasser and Theurer and Jackson Company ballast tampers that are used for tangent and curved track, switches, and joints are listed in Table 6.5. Models of other tamping machines for which specifications were not obtained, are listed in Table 6.6. The data included in Table 6.5 are 1) weight of equipment, 2) production travel speed, 3) tamping tool characteristics such as location, number of tools, amplitude, frequency, rated static and dynamic forces, and 4) surfacing and lining operations used. The lining and leveling systems in use are described by Diaz and Janderes (Ref. 223) and should be referred to for further details. Variations in shape and length of tamping tools are also noted to occur.

The tamping operation can be used in conjunction with or independently of surface and lining operations. Present day tamping equipment all utilize the vibratory squeeze principle with either a synchronous or a non-synchronous squeeze action. In the synchronous squeeze, equal system pressure is applied to each opposing pair of vibrating tines which move the same distance until the desired squeeze pressure is reached. For the non-synchronous squeeze, the hydraulic pressure in the opposing tines may be different and is a function of the ballast resistance. When the pre-established pressure is achieved in one tine the movement ceases and the opposing tine squeezes until the same pressure is reached. Plasser and Theurer (Ref. 208) utilize the non-synchronous concepts while Canron (Ref. 207) uses the synchronous squeeze action. Jackson (Ref. 222) appears to use a concept similar to Canron.

Table 6

Manufacturer	Model No.	Production
1. (Ref. 207)		
(a) Canron	JP	60 joints/hr
(b) Tamper	VI-Vibra-tool	---
(c) Tamper	SVI-Switch Vibratool	---
(d) Tamper	EAS Switch Electromatic Mark I	---
(e) Tamper	EAS Switch Electromatic Mark II	---
(f) Canron	Electromatic Mark I	1500 ft/hr
(g) Canron	Electromatic Mark II	1500 ft/hr
(h) Canron	E-JH Mark II	---
(i) Canron	Mark III T 2233 DAG	---
(j) Matisa	B200 series 201 202 204 220,230,240	2300 to 3950 ft/hr " "
2. (Ref. 222)		
(a) Jackson	Utility tamper	---
(b) Jackson	2300	750 ft/hr
(c) Jackson	Automatic tamper 4500	1000 to 1200 ft/hr

.5. Ballast Tamping Equipment

Type Track	Amplitude	Frequency (Hz)	Leveling and Lining Operations
Joints and correction, low spots	Positive 3/8 in.	53.3	Automatic surfacing with infrared beam
Tangent and curved	---	53.3	With or without optional wire
Tangent, curved and switches	---	53.3	Reference device for leveling & lining and crosslevel indicator
Tangent, curved and switches	Positive 3/8 in.	53.3	Optional delta surfacing device infrared beam
Turnout, crossover and ladder track	Positive 3/8 in.	53.3	Optional delta surfacing device infrared beam
Tangent and curved	Positive 3/8 in.	53.3	Optional delta surfacing device infrared beam
Tangent and curved	Positive 3/8 in.	53.3	Optional delta surfacing device infrared beam
Tangent and curved	---	---	Optional delta leveling system and autograph liner
Switches lift & line curved with check rails	---	---	Autograph liner
Tangent and curved " " "	---	---	3 point leveling system; 4 point lining; automatic
Tangent and curved	---	70	Light
Tangent, curved, switches and turnouts	---	75	Light
Tangent and curved	---	75	Crossleveling electro-optic servo-controlled hydraulic system

Table 6.5.

Manufacturer	Model No.	Production
(d) Jackson	5000	1600 ft/hr
(e) Jackson	6000	---
3. (Ref. 208)		
(a) Plasser & Theurer	Roadmaster Universal 06-165LC	---
(b) Plasser & Theurer	R.U. 06-16	---
(c) Plasser & Theurer	Assistant Roadmaster	---
(d) Plasser & Theurer	UYT-2W75	---
(e) Plasser & Theurer	UJT-06-16	---
(f) Plasser & Theurer	Universal 07-16 DM	---
(g) Plasser & Theurer	Duomatic 07-32 DN	---
(h) Plasser & Theurer	Plassermatic 07-275 DN (PLM 07-275 DN or PLM 07-275)	---
(i) Plasser & Theurer	Quatromatic	Up to 5000 ft/hr

Ballast Tamping Equipment (continued)

Type Track	Amplitude	Frequency (Hz)	Tamping and Lining Operations
Tangent and curved	---	75	Built-in truss beam concept, automatic, optional lightning buggie
Tangent and curved	---	75	" " " " "
Tangent and curved	---	--	Automatic lift and leveling, automatic 2-chord and laser beam lining
Tangent and curved	---	--	" " " " "
Tangent and curved	---	--	-----
Yard, switch & spot	---	--	Lifting by wire surfacing device and shadow board control; leveling with crosslevel pendulum
Universal joint, spot & tandem tamper	---	--	" " " " "
Tangent and curved	---	--	Laser or optic leveling/lining units (single chord measuring system)
Tangent and curved	---	--	" " " " "
Tangent and curved, switches & crossings	10 mm	35	Infrared beam leveling
Tangent and curved	---	--	Laser or optic leveling/lining units (single chord measuring system)

Table 6.6. Additional Ballast Tamping Equipment

Other tamping equipment for which information or specification sheets were not obtained.

MANUFACTURER	MODEL	COMMENTS
1. Matisa (Ref. 207)	BMNRI 85	—
2. Robel	Supermat 62.43	—
3. Plasser & Theurer (Ref. 208)		
a)	Universal Duomatic 06-32	Tamper-Liner
b)	Universal Yardmaster UYM 8-L	Switch production tamping, 2 4 tool units, can surface & line a switch in 30-45 min.
c)	General Roadmaster GRM 16C	Tamps switch, frog, tangent & curved track; 2 8-tool or 6-tool heads
d)	Tie Tamper PTT-16	Spot, tamper & tandem tamping, 16 tool
e)	Roadmaster Special	Tangent & curve, single chord lining & leveling, double output, max. 2200 m/hr.
f)	07-Super 32 RSV	Sleeper end consolidating plates, also all round tamping system
g)	WE 75	1 tamping unit, shoulder & crib, 1962.
h)	WE 275	2 tamping unit, shoulder & crib, 1962.
i)	Main Liner Universal 06 Series	—
j)	Beaver 4-73 73 800	Tiltable tools, for industrial railways or city transport
k)	Plassermatic 06-8	—

The main differences in the tamping operation between manufacturers occur in vibrating frequency and in amplitude of vibration of the tamping tines. The frequency is 35, 53.5, and 70 Hz and the amplitude is 10 mm (0.39 in.), 3/8 in., and 1/8 in. for Plasser and Theurer, Canron and Jackson, respectively (Table 6.5). Other differences which exist are whether two or three, or four ties are tamped simultaneously, the addition of tie-end tamping or the addition of shoulder compacting plates.

Cassidy (Ref. 212) identifies some specific tamper design characteristics which should be further described and quantified:

1. The bounds of the volume of the disturbed ballast.

2. The definition of squeeze pressures, which is not well defined in the company brochures. Some specifics to be quantified are:

   a. Magnitude of maximum forces, bearing areas, and maximum pressures,

   b. Force vs. time (or distance vs. time) curve.

   c. The means by which the number of squeezes on a given tie is determined.

3. Vibration characteristics including whether the vibration is force controlled or distance controlled, and the exact means by which this is obtained.

The tamping procedure used by Canron, illustrating the vibratory squeeze principle, is shown in Fig. 6.1. The tines are set into vibration before insertion in the ballast. The steps in tamping are (Ref. 207):

1. Downfeed: In addition to the free fall weight of the tamping units, downthrust pressure on the ballast of up to 2,000 psi (140.6 kg/sq cm) is available. Lower limit switches stop downfeed at a preselected depth which is adjustable down to 6 in. (152 mm) below the bottom of the tie.

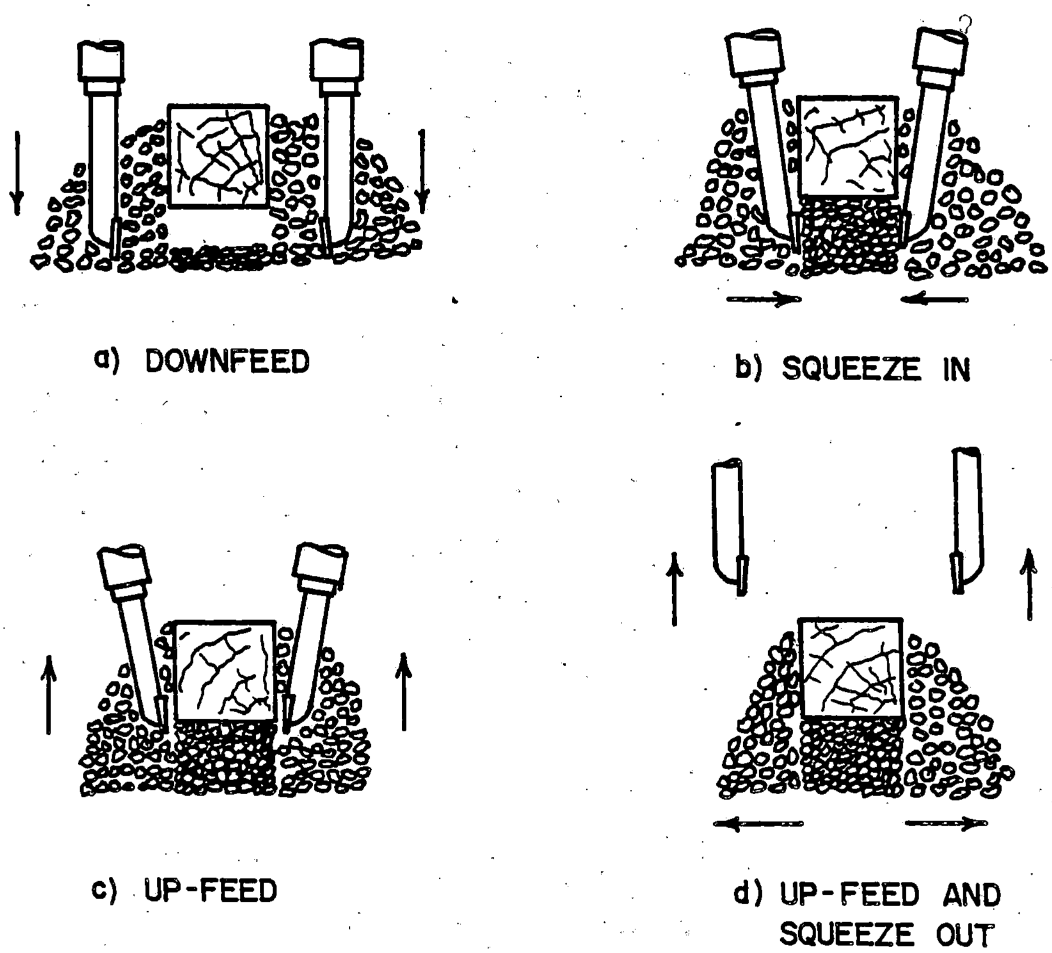


Figure 6.1. Pinch and Squeeze Method (Ref. 207)

2. Squeeze-in: The rate of squeeze-in is variable and is pre-set to suit ballast conditions. Interconnections in the hydraulic squeeze circuit assure uniform squeeze pressure. Squeeze-in stops when a pre-set hydraulic pressure is reached due to pressure build-up between opposing tools. Interconnection of hydraulic squeeze circuits provides uniform squeezing along the entire tamped area of the tie.

3. Up-feed: The tamping tools during the initial portion of the up-feed retract upward along the side of the tie, so that the crib ballast will fall against the sides of the tie to retain ballast in the tamped position. The tamping tools return to the upper limit prepared for next cycle.

For low lifts, smoothing, spot track repair work, and tamping in confined areas such as switches, small portable vibratory tie tampers have been used. Jackson (Ref. 222), Modern Track Machinery, Inc. (215), and Wacker (Ref. 224) produce several hand tie tampers. These tampers are not production oriented for large jobs of major reballasting or out-of-face trackwork. A comparison of their effectiveness in ballast tamping compared to large tamping machines is not known.

6.7 BALLAST COMPACTION

The final operation in major out-of-face trackwork after lining-leveling and tamping is compaction of the crib and shoulder by the use of ballast compactors. This practice has been frequently used on European railroads, whereas in the U.S. it has primarily been used on a preliminary trail basis.

In operation , crib compaction plates are applied to the surface of the ballast in the crib area where the tamping tines are inserted. At the same time, another set of compaction plates, with or without an attached shoulder pressure plate, compacts the ballast at the tie end or on the shoulder. The purpose of the shoulder plates is to prevent the lateral flow of ballast during compaction operations, particularly in the case of narrow shoulders. Vibration is produced by either of two methods:

1. a rotating eccentric mass generating a constant dynamic force for a given frequency, or

2. an eccentric shaft producing a constant amplitude of oscillation.

The lack of experimental data prohibits a conclusion as to which is the more effective means for ballast compaction.

Two manufacturers presently produce ballast compactors. The machines developed by Matisa Materiel Industriel are handled in the U.S. by CANRON, formerly known as Tamper. Other machines have been developed by Plasser and Theurer together with Windhoff. Specifications for representative ballast compactors are given in Table 6.7. Included are characteristics of the compaction plates, such as type, number, shape, amount of down pressure and location, production (dependent upon cycle time and tie spacing), amplitude and frequency of vibration, as well as the rated static and dynamic forces. Major differences in equipment occur with the type and outputs of the vibratory motors. The reliability of the rated dynamic force is questionable, since this value depends upon the physical state of the ballast and the subsequent interaction with the vibrating compaction plates. Excluded from the list are one-of-a-kind machines developed by railroad companies for experimental purposes.

Plasser and Theurer first built a combined tamping and compacting machine, a VKR03 with crib compacting units, in 1958. The VKR03 was the forerunner of the modern Plasser & Theurer combined leveling, tamping, lining, and compacting machines (Ref. 227).

Table 6.7. Ballast Compaction Equipment

Reference	Manufacturer	Model No.	Production (ft/hr)	Type Vibration	Amplitude	Frequency (Hz)	Down Pressure	Rated Force (lb) per plate Static	Dynamic
225	Plasser and Theurer	VDM800U	2300 to 2950	Vibratory motor, crib - crank rockers, shoulder and tie end - shaft type eccentric drive	0.138 in. constant	38.5 (crib) 35 (tie end and shoulder)	--	Crib- 1730 Tie end- 2200 Shoulder- 2200	1100 1100 2420
208	Plasser and Theurer	CPM-800-R	---	" , assume same	" , assume same	" assume same	--	---	---
207	Tamper	CSC	---	Vibratory motor	---	---	--	---	---
226	Plasser and Theurer	BKV142 Mainliner Universal 06-16CTM	---	(Same as VDM 800 U)	---	---	--	---	---
226	Windhoff	BV102 BV1025 BV103 FD41	--- --- --- ---	--- Vibratory motor "	--- --- --- ---	--- 48 (tie end) " "	-- -- -- --	--- --- --- ---	--- 2200 1540 to 3740
208	Plasser and Theurer	Dyn. Track Stabilizer	5575	Vibratory motor, horizontal vibration & vertical load	---	0 to 50	0 to 64 kips	0 to 24,000	0 to 64,000
207	Matisa	Compactors D-912 D-9 D-912R	1640 to 2620	Eccentric vibration	---	50	Adjustable	---	Crib up to 5500 Shoulder up to 6600
208	Plasser and Theurer Windhoff	901SW	2790	Assume vibratory motor	---	---	--	---	---

Table 6.7. Ballast Compaction Equipment (continued)

	Crib Plates				Shoulder or Tie End Plates			
Reference	No. Units	No. Plates Per Unit	Contact Area Per Unit	Head Shape	Shoulder Pressure Plates	Length	Width	Contact Area Per Plate
225	2	4	115 in.^{2}	V-Shape	With or Without	52.8 in.	9.8 in.	1039 in.^{2} 403 in.^{2}
208	2	4	---	Spherical	" Automatic Adjust any slope	54 in.	8 in.	---
207	2	4	---		Manual Adjust any slope	---	---	---
226	2	4	---	---	---	---	---	---
226	2	2	---	Spherical	No	---	---	---
	2	4	---	---	Manual adjust slope	---	---	---
	2	4	---	---	" "	---	---	---
	2	2	---	---	No	---	---	---
208	2	2	---	---	---	---	---	---
207	2	4	---	V-Shape	No	---	---	---
208	2	4	---	Spherical	No	---	---	---

A review of manufacturers literature on ballast compactors produced the following as some of the claimed advantages for use of this machine in conjunction with normal track maintenance operations.

1. Increases ballast density in the cribs and increases bearing support under the tie following tamping.

2. Increases lateral and longitudinal track resistance and track stability, both immediately and long term, which prvides a greater security against track buckling and sun kinks.

3. Increases friction against the sides and ends of the tie.

4. Increases the lifetime of cross level, line, and surface, so that maintenance intervals are increased.

5. Shoulder compaction fills voids at the end of the tie.

6. Provides a uniform bearing surface (pressure distribution) under the tie.

7. Permits higher train speed immediately after compaction compared to after tamping only.

8. Reduces the number of loose ties, which are a cause of track deterioration including rail wear and fractures.

According to Powell (Ref. 228), crib and shoulder compaction is effective in increasing the lateral resistance after tamping, only if a sufficient amount of ballast is present in the crib, i.e., a full crib. CNR has made a preliminary review of the cost and benefits of ballast compactors (Ref. 192). Increased costs are incurred in owning, operating, and maintaining ballast compactors, and in the additional ballast material required to fill the cribs. Whether or not the long-term benefits justify these costs has not yet been established.

Some of the claimed advantages are based on experimental findings. However, the confirmation is primarily limited to short term benefits. The degree of increase in the maintenance cycle is still indeterminate, although Birmann and Cabos (Ref. 203) calculate this to be as much as 33%. Other advantages listed, such as providing uniform support and resistance, have not been verified. The conditions required to yield the highest degree of compaction, particularly under the tie, have also not been determined.

Plasser and Theurer has recently developed the dynamic track stabilizer which is distinctly different from previously described machines. Instead of compacting the ballast in the crib and shoulder area by using surface plates, the whole track is vibrated horizontally with a vertical load applied to the rail. The machine has two counter-rotating eccentric masses oriented with their shafts parallel to the rail. The arrangement of the eccentric masses is such that the vertical component of vibration is cancelled out and the horizontal of the vibratory force is in the direction parallel to the ties. Although the dynamic force is adjustable by varying the position of the eccentric masses, this is usually fixed and the dynamic force changed only by changing the vibration frequency, with the range of about 18 to 40 Hz.

The mechanism of clamping the dynamic force generating component to the rails is by means of rollers so that the machine can travel along the rail during vibration. A typical travel speed is one kilometer per hour. This approach is distinctly different from the crib and shoulder compaction machine which is stationary during the period of ballast compaction. A vertical force is simultaneously applied to the rail using the weight of the machine as a reaction. The magnitude of the vertical force is controlled on the basis of the vertical displacement. Using a light beam system like that on tamping machines, the vertical displacement is controlled in reference to the rail positions at the end of the machine. A maximum differential vertical displacement between the vibrator and the references at the end of the machine is selected, not to exceed 20 mm. The vertical force is adjusted automatically until the displacement reaches this limit or until the maximum vertical force is achieved, the latter being equivalent to a light axle loading on the track vehicle. Separate controls are provided on each side of the machine so that deflection of each rail is independently maintained. Because this machine is designed to be used after a tamping operation, the control of the vertical displacement in the manner described is necessary to maintain surface. Otherwise, because of the variable resistance of the ballast to vibration, the use of this machine with a constant vertical force would reintroduce irregularities in the track geometry.

Chapters 7 and 8 review available results from studies of ballast compactors, demonstrating the effects of the test parameters and the effectiveness of the procedures on track performance.

7. RELATIONSHIP OF BALLAST COMPACTION TO TRACK PERFORMANCE

In this chapter, various methods currently being used to measure track performance are discussed. This chapter will also identify the current trends and methodology being employed in performance prediction, and the problems involved with such predictions. The principal emphasis, however, will be on the evaluation of the effects of ballast conditions and compaction on track performance based on a review of available literature.

7.1 TRACK PERFORMANCE MEASUREMENT

The currently available or most commonly used techniques of quantifying the physical states of the track include track settlement surveys, tie displacement tests, track modulus tests, track geometry surveys, and joint profile surveys.

Track Settlement Survey. A track settlement survey measures both differential and overall track settlement with respect to a fixed reference. Profiles of rails are usually obtained at an appropriate station interval using the standard physical surveying equipment such as a transit or level with rod and tape measure. Measurements are made periodically over time after maintenance, and the profile immediately after tamping and before any traffic, is generally used as a reference. The track settlement requires a long period of observation, since the effects on the track settlement of various track components and of the construction and maintenance are generally not expected to become evident until the accumulation of significant traffic. Furthermore the evaluation of track settlement is not a sufficient means to assess the effects of ballast compaction because settlement also depends on various other parameters of equal or greater influence. For example, caution should be exercised in interpretation of results when the track has a subgrade layer which is susceptible to environmental factors such as moisture changes, and frost.

Tie Displacement Tests. Tie displacement tests involve measurement of the resistance of a single tie or multi-tie panel subjected to either lateral or longitudinal force. This type of test has been used both to evaluate the ballast state and to assess track stability. The lateral tie pull or push test (LTPT) on a single tie, in particular, has been widely used throughout the world in evaluating effectiveness of various maintenance operations including crib ballast compaction. The common procedures consist of removing the tie fastenings under both rails to isolate a selected tie from the surrounding track structure, and displacing the tie at a certain rate of loading or displacement while taking measurements of the applied force and resulting tie displacement. Forces at 2 or 4 mm displacement are commonly used as typical resistance values. Displacements are usually made by either pulling or pushing the tie with a hydraulic jack reacting against the rail. Figure 7.1 illustrates various approaches currently being used in the single-tie tests.

Some investigators favor testing with a multiple-tie panel instead of single-tie testing. Their arguments are: 1) a single tie completely isolated from the surrounding track would not represent the lateral restraint of the actual track under traffic, 2) a single tie would not simulate the effect of pressure overlap within the ballast section caused by adjacent ties and the effects and motions of crib ballast, and 3) the resistance provided by the tie-fastenings and rails could not be included in the single-tie tests.

In the multiple-tie tests, a panel of track consisting of several ties is separated from the rest, without removing the tie fastenings, and then displaced as a whole. The number of ties involved in the test varies widely.

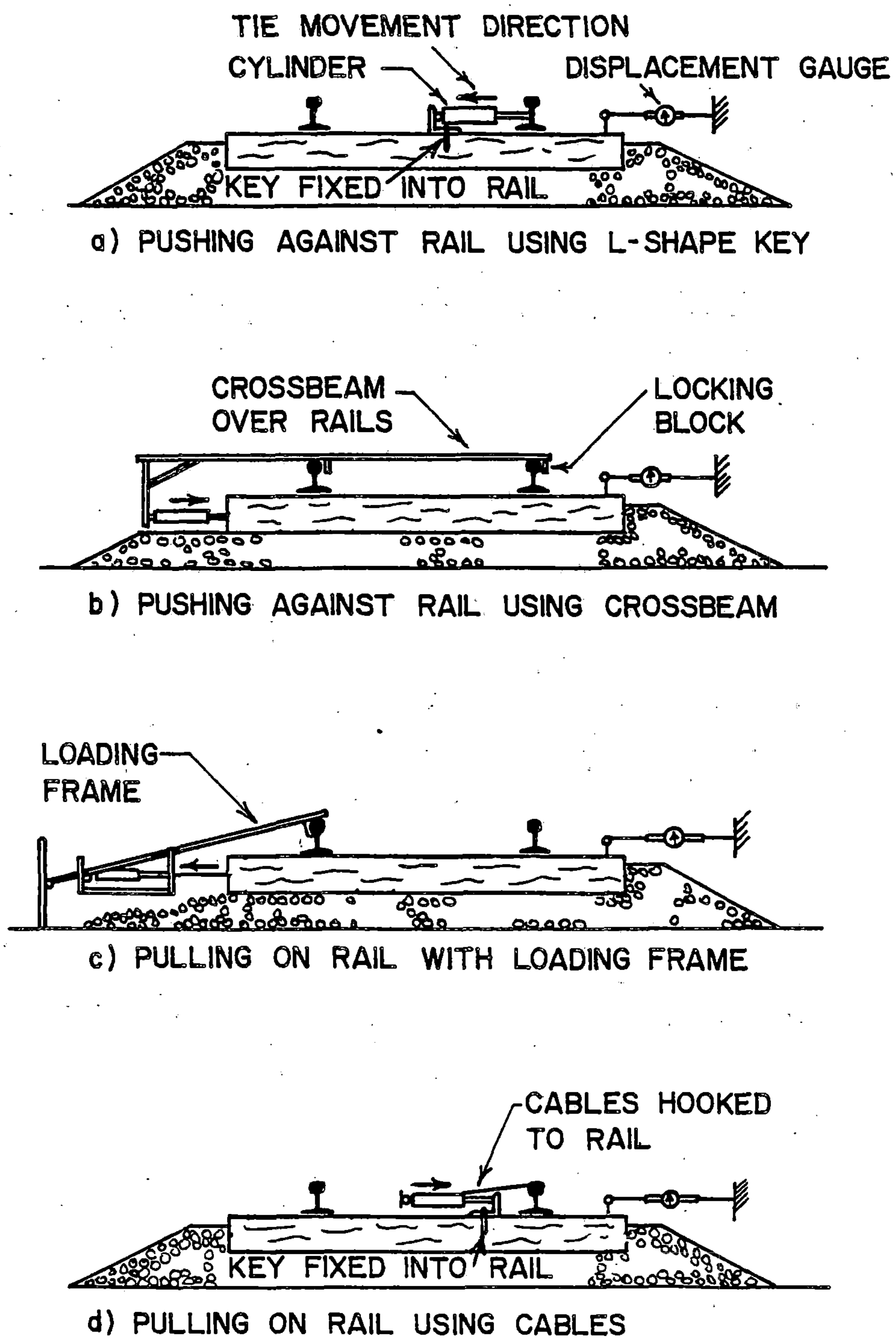


Figure 7.1. Schematic of Various Techniques Being Used in the Lateral Tie Displacement Test

Magee (Ref. 229) used two ties in track loading studies on polymer-stabilized ballast. At the University of Illinois (Ref. 230), a 3-tie panel was used in the laboratory simulation. In Japan, studies of lateral resistance of concrete ties were performed using 4 or more ties (Ref. 231). The British Transport Commission (Ref. 232) reported the use of a 6-tie-track section, while a 9-tie section of track was used in France (Ref. 233). Reissberger (Ref. 202) also reported wide uses of a 7-tie panel in other European countries. In the U.S., larger test sections seem to be favored. The 1972 AREA ballast test proposal (Ref. 2) specified a minimum track length of six feet. Track panels of 39-ft length were used in the studies on the FRA ballast compactor (Ref. 234). A 39-ft panel represents the entire length of a standard rail commonly used in the U.S. The general set up of these tests is illustrated in Fig. 7.2.

Regardless of the number of ties involved with the test, most of the techniques currently used create an undesired vertical force component on the tie when the lateral force is applied. This vertical force tends to lift up or force down the tie(s) and thus change the lateral resistance. Also, the disturbance during tie-fastener removal and attachment of the loading scheme to the tie in the case of a single-tie test may be significant.

The usefulness of the LTPT method with an unloaded tie as a measure of the lateral restraint of a track is also questionable. First of all, the contributions of the crib, the shoulder and the tie bottom to the total lateral resistance is expected to be quite different for an unloaded tie than for a tie carrying a portion of a vertical axle load. It is also well recognized (Refs. 11, 235) that the track under traffic tends to lift some distance away from the rolling wheel. This unloading effect is not considered in the lateral tie pull test.

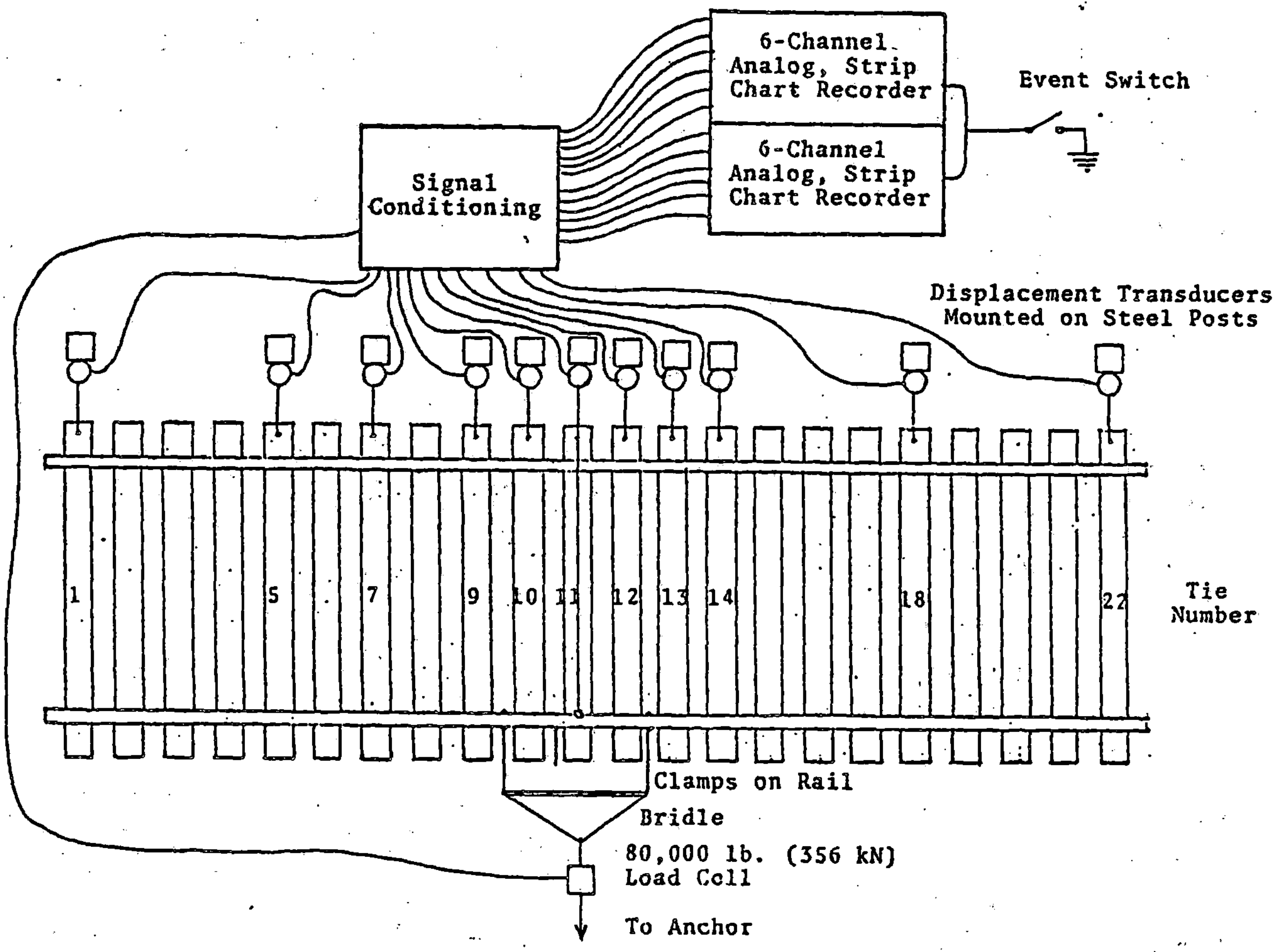


Figure 7.2. Schematic of 39-ft Panel Test (Ref. 234)

Further details and a critique of the LTPT method of track performance measurement will be described later.

A longitudinal tie displacement test has been used also for evaluating the resistance of track against longitudinal forces transmitted from the rails such as those from braking and accelerating, and from temperature-induced expansion and contraction (Refs. 202, 205). To conduct this test, selected ties are separated from the track and pushed in the longitudinal direction. A single tie is usually involved in the test using a test equipment configuration such as that in Fig. 7.3.

Track Modulus Test. The track modulus test is a measure of vertical track stiffness under load. Track modulus is defined as the load per unit length of rail required to depress the rail one unit of displacement. Generally, a specified length of track is loaded with a static vertical load, and the corresponding deflection measured using a surveyor's transit. An example of such a test set up is shown in Fig. 7.4. An alternative method of measuring track modulus would be to use a long flat car equipped with an unloaded floating axle at the midpoint. The sections of the car over the trucks are loaded. The difference in track deflection between the unloaded and loaded axles is continuously recorded as the car is towed along the track. With the measured force and deflection, track modulus is often obtained by solving the equation for track deflection that has been derived from the analysis of a continuous beam resting on a continuous elastic support based on the work of Talbot (Ref. 6).

The track modulus test has been long used to obtain design information for the track system as well as to evaluate the track performance after construction and maintenance. However, the test results do not distinguish the

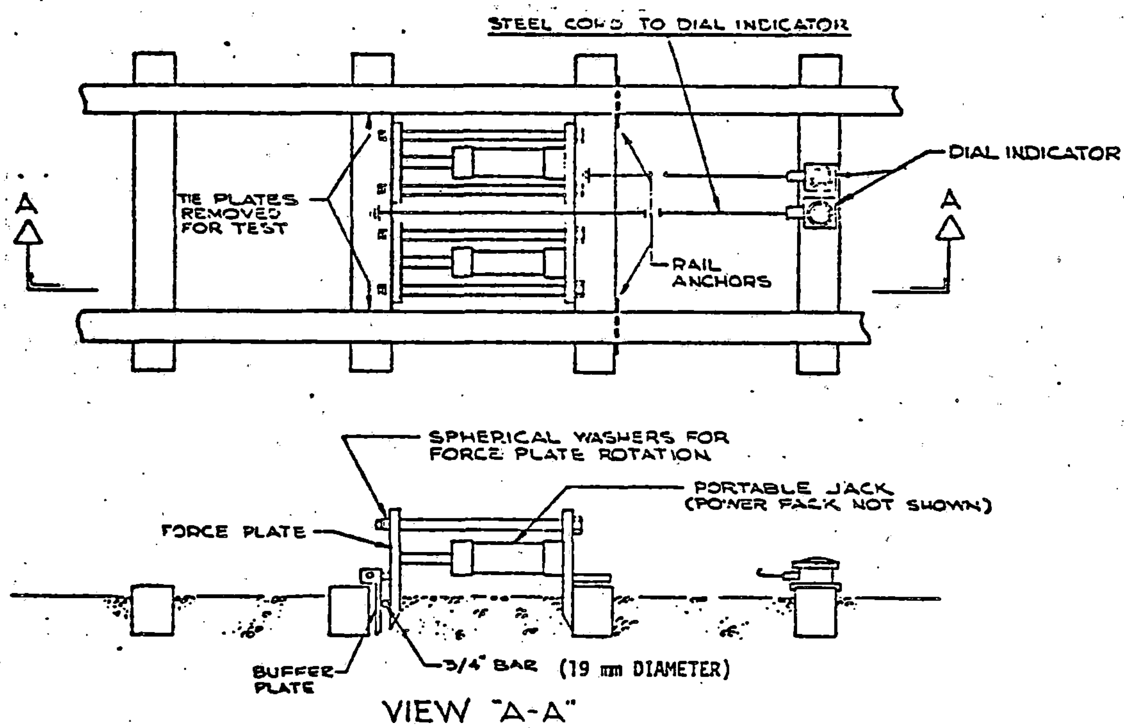


Figure 7.3. Schematic of Longitudinal Tie Resistance Test (Ref. 205)

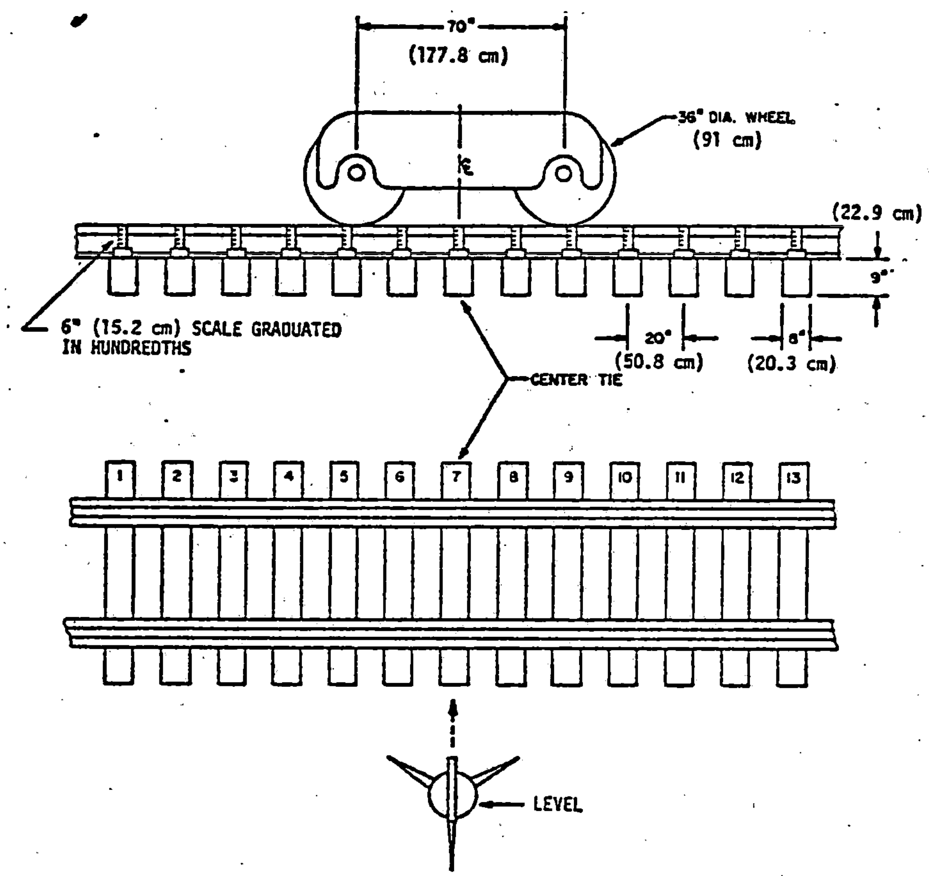


Figure 7.4. Schematic of Track Modulus Measurement (Ref. 205)

importance of contribution of various track components, such as the spacing, dimensions, or quality of a tie, or the quality, depth and degree of compaction of the ballast and subgrade layers.

Use of a vertical load-deflection test with a single tie isolated from the surrounding track structure has also been reported in the literature (Ref. 202). However, this test seems to be mainly related to studying the ballast resistance under the tie.

<u>Track Geometry Surveying</u>. Track geometry surveying with a recording car is one of the most popular methods used for determining the quality and performance of track. The parameters describing track geometry such as gage, line, and surface are usually measured with an inspection vehicle, and continuously recorded in strip chart form. The measure of such parameters and their changes is a direct indication of track performance, especially the riding quality, and has been widely used in the railroad for establishing needs for maintenance work. The method, involving a vehicle travelling on the actual in-service track at a speed comparable to the actual traffic, has an advantage not only of representing the track performance more realistically than any other methods previously described, but the entire length of a track section is surveyed rather than just a few discrete points. Also, the method can be used without disturbing the track and interrupting traffic, and it can be done very rapidly without any significant preparation.

<u>Dynamic Displacement</u>. Tests to measure the dynamic vertical displacement at rail joints have also been used in assessing the track performance. An example of a test configuration is shown in Fig. 7.5. This is of interest because changes in track profile at joints in bolted rails contribute significantly to rail and riding quality deterioration. An alternative method for

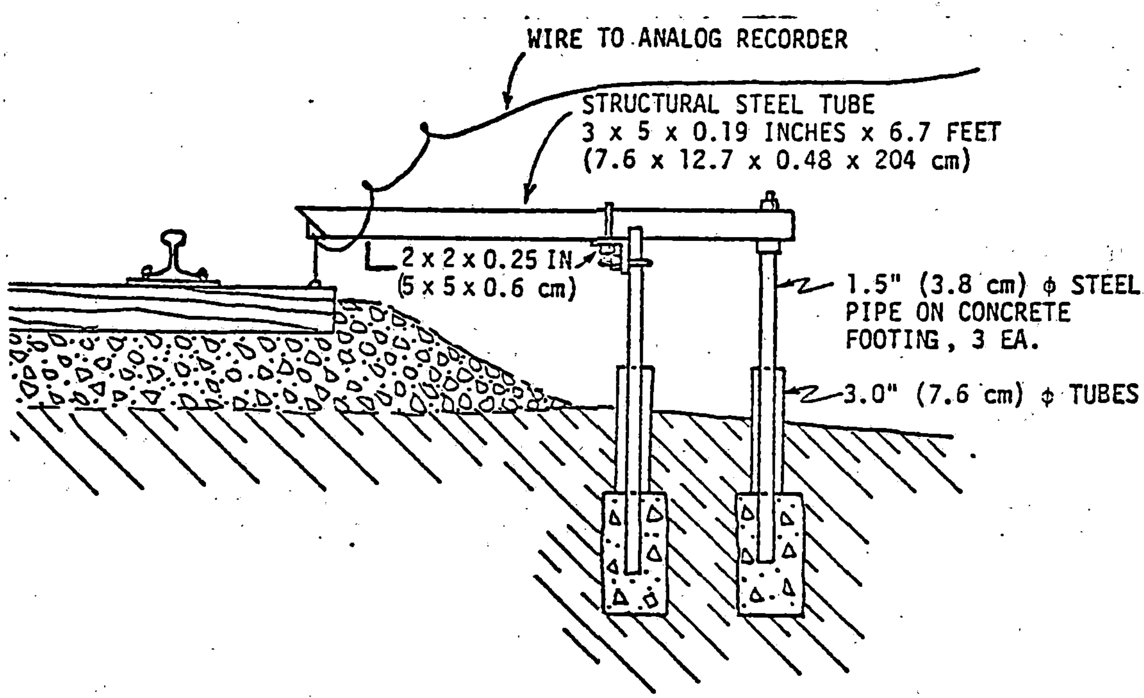


Figure 7.5. Schematic Installation of Displacement Transducer to Measure the Settlement at Joint (Ref. 236)

measuring dynamic deflection under moving traffic load is shown in Fig. 7.6.

All of the methods described above measure only a qualitative picture of overall track performance, even though methodology has been developed for quantifying the results of most of the tests. There is at the present time no way of relating the test results to an index that could uniquely define the level of track performance.

7.2 APPROACH TO PERFORMANCE PREDICTION

Prediction of track performance for a given set of conditions is necessary to provide rational guidelines for track design, construction, and maintenance. To do this properly, the significance of each track component should be quantified.

There are currently two approaches for track performance prediction. One deals with the structural response of track (Ref. 1,210) and the other with the physical condition (Ref. 238). The former approach relates the measured or calculated track response under different loading conditions to the long-term structural behavior or performance of the track. The latter determines track quality or performance based on track geometry or dimensional changes over a period of time and relates these to the track system parameters.

Structural Track Response. To use the track structural response approach, a complete understanding of track behavior under traffic loading and environmental changes has to be achieved. Then an extensive data base is required to develop the relationships between the dynamic track response and the long-term performance. Understanding of the track behavior at the present time is limited due to the complexities inherent not only in the track system, but in the track loadings. Little meaningful data of the track response is available from field measurements, and the effects of each component of the track system on the measured responses are often not distinguished. In addition, it is also

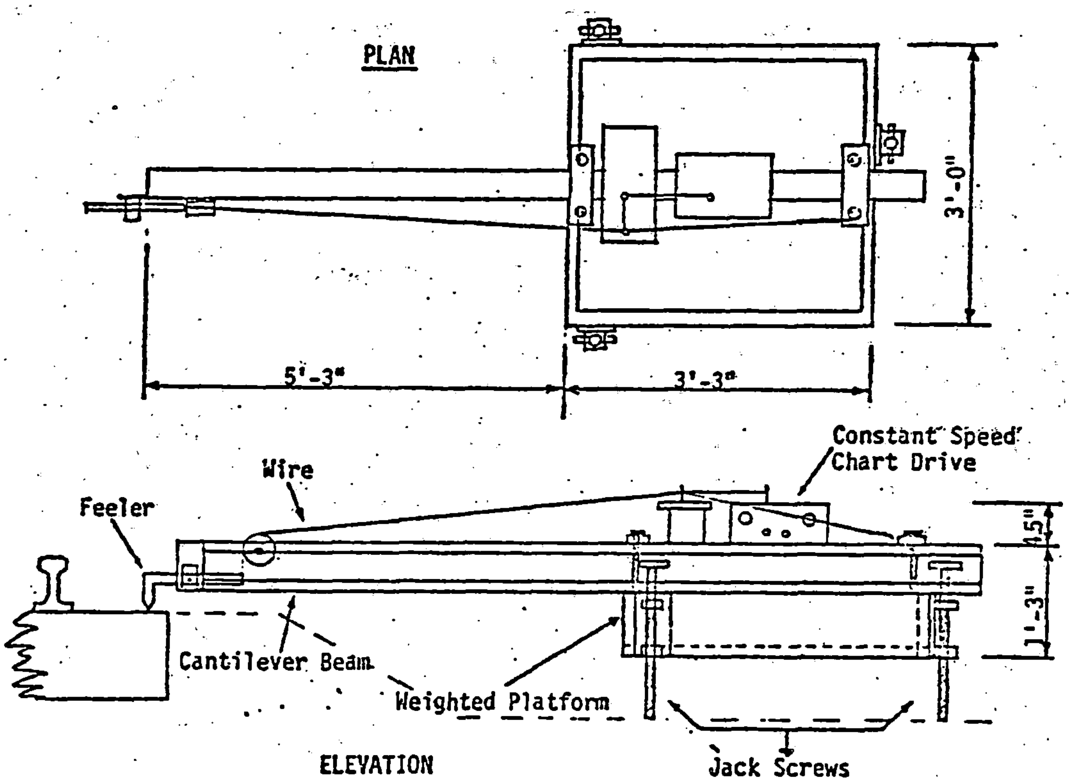


Figure 7.6. Schematic of Dynamic Settlement Measurements (Ref. 237)

difficult to estimate the variations of the structural response expected with time and environmental changes. There have been no experimental studies of track response with adequate consideration to such factors as material property changes with time, different loading spectra, and environmental effects.

As a result of the limitations of available experimental investigations, analytical models of the track structure are very useful in predicting track system response and, particularly, in evaluating the rôle of individual components. Such models may ultimately provide analytical tools for predicting track performance, and therefore aid in the technical and economical evaluation of track. Some of these analytical methods are described in Section 2.

Once the structural response and the effects of components are identified, they must then be related to a track performance index or rating. That requires extensive data obtained from the long-term observation of track performance under various conditions, and an indexing or rating system of track performance which could possibly integrate the various criteria.

Borrowing the concepts and approaches that have been developed for the prediction of highway and airfield performance, Robnett et al., (Ref. 1) suggests the use of a "transfer function" for relating the structural response of track to "performance."

In case of airfield pavements, the service life of the pavement has been successfully related to the early life deflections based on road test results (Ref. 239). However, in the case of railroad track, the development of such a transfer function is still in the conceptual stage. The more complicated interaction between larger numbers of structural components in track than in pavements, as well as the lack of information required to develop a suitable transfer function, have hampered the progress. No unique system for quantitative indexing or rating the performance level of track has been developed yet. Instead, the current approach is to employ a set of criteria that each of the components has to meet, and to estimate the level of track performance relative to how well each component satisfied these pre-determined criteria.

Robnett, et al. (Ref. 1) suggested using allowable stress and strain levels, and determining the degree of expected distress development by comparing the track response with the allowable level. Meanwhile Prause, et al. (Ref. 210) recommended using failure of components as the criterion. They suggested that by establishing relations between the material properties and the track system response and performance leading to an identification of the cause and development of failure, a track performance index could be developed. Such efforts require extensive surveys and review of the past failure history, as well as model studies of track response. Industry specifications also have to be considered to establish proper failure criteria.

In summary, the transfer functions for railroad track system performance prediction are still in the conceptual stage. Understanding of track behavior at the moment is very limited. Many parameters interact in a complex way under traffic and environmental loads. Even the critical response features related to performance have not yet been identified. Also, there is lack of information on track performance, in part, because of inadequate methods for track and measurement performance ranking systems. Furthermore the railroad industry often does not maintain adequate documentation of in-service conditions, such as maintenance and traffic history, which is absolutely necessary to develop a meaningful transfer function.

Recently an attempt has been made to investigate the performance of various types of track components under controlled field testing conditions. A facility for accelerated service testing (FAST), consisting of various types of track structure, including rail, ties, ballast and fasteners, has been constructed at the US DOT Transportation Test Center in Colorado to gather information on track and train performance. Although these tests are expected to provide useful information for understanding track performance, the data obtained from FAST may not cover a sufficiently broad spectra of track conditions needed for development of transfer functions.

Track Quality Index. The track quality index approach, which has evolved from the needs for establishing rational guidelines for a track maintenance program, ranks the performance based on the changes in the track geometry parameters such as gage, twist, surface, superelevation, and alignment (Ref. 238). The physical condition of the track has been one of the most important factors determining the maintenance needs because the track performance is closely related to the track geometry. For example, rough track resulting from unstable track geometry increases the hunting of trucks, sway of cars and wheel-rail forces. Such adverse effects of track geometry distortion obviously limit the performance of track by causing poor riding quality, increased deterioration of locomotives, cars, and lading, reduced operating speed, and increased possibility of derailments.

The basic approach for rating track based on measurements by a track geometry inspection car is to determine how much the measured track geometry deviates from the pre-established tolerance limits of the various track geometry parameters. In deriving the track quality index, various parameters are assigned different weighting factors depending upon the nature and characteristics of the track, and are summed into an index. The tolerance deviations may be subdivided into several threshold tolerance levels to which different weighting factors can again be assigned, in order to reflect both the frequency and the magnitude of tolerance deviations.

The track quality index is an indicator of the track conditions at the time of track inspection. However, when data are compiled over a sufficient period of time, correlation can be made between track deterioration rate and track operating conditions such as speed and traffic density, for the purpose of track performance prediction. If such correlations are obtained for a variety of track structures, then an approximate relationship between performance and track components can be established. The biggest problem with this approach is the determination of the existing ballast and subgrade conditions.

Presently, the track inspection cars with modern high-speed inspecting equipment, can measure not only the track geometry, but ride quality, car rocking, drawbar pull, acceleration on the axle, and any other data which are useful in determining track performance. By integrating all this information, a more meaningful track performance index might be generated. For example, by comparing the track quality index for a particular segment to that of the total system, a more efficient track maintenance program can be established based on priority needs. Also, the ratio of gross ton miles for a particular line to system gross ton miles, or the ratio of net value of goods carried by division or line per unit time to system net income per unit time can be integrated with the track quality index to come up with overall priority-quality-profit rating system.

The approach of track performance prediction is again based on the arbitrarily established tolerance limits and weighting scale in considering various factors influencing track performance. For example, even the track geometry tolerance limits vary widely among the railroads as illustrated in Table 7.1.

Table 7.1 Track Geometry Tolerance Limits for Various Railroads (Refs. 10, 240, 238)

Track Geometry Parameters	FRA (for Class 6 Track)	British Railways	Deutsche Bundesbahn	Japanese National Railways
Gage	For tangent track min. 4 ft 8 in. max. 4 ft 8-3/4 in. For curved track min. 4 ft 8 in. max. 4 ft 9 in.	Maximum 1/8 in. open or tight	For mainlines min. 4 ft 8-1/4 in. max. 4 ft 9-1/2 in. For branch lines max. 4 ft 9-1/2 in.	max. 9/32 in. open max. 5/32 in. tight
Distortion		1:500	1:500 (High Speed) 1:300 (Normal)	
Surface	1/2 in. in either 31 ft or 62 ft, depending on track conditions	3/8 in. in 31 ft	0.24 in. in 33 ft	For mainlines 0.35 in. For branch lines 0.43 in. (base not known)
Alignment	For tangent 1/2 in. on 62 ft cord For curve 3/8 in. on 62 ft cord	$\pm$ 1/8 in. based on 33 ft cord	$\pm$ 0.17 in.	For tangent or curve over 800 m rad. $\pm$ 0.28 in. For curve less than 800 m rad. $\pm$ 0.35 in.
Super-elevation	5/8 in.	$\pm$ 3/8 in.	$\pm$ 2 mm	For tangent or curve over 800 m rad. $\pm$ 8 mm For curve less than 800 m rad. $\pm$ 9 mm

However, this method is a simple means of estimating the track performance based on end-results of track response. In other words, unlike the structural response approach, the index approach may not require a complete knowledge of track behavior under different loading conditions.

7.3 EFFECTS OF BALLAST CONDITIONS ON PERFORMANCE

There are numerous parameters influencing various aspects of track performance. Since the track system consists of many different components which contribute to the overall performance, and the term "track performance" includes a very broad spectra of different criteria ranging from the physical appearance and stability of track to economics of track operation, it may be impossible to list all the factors influencing track performance in a simple way. However, the factors may be conveniently categorized as track structure, traffic, and environmental parameters. In the track structure category, design, construction, and present conditions of each track component such as rail, tie-plate, anchoring, tie, ballast, and subgrade are included. Traffic conditions include traffic loads and density, speed, and types of service such as passenger or freight. Temperature and weather changes, as well as nature and characteristics of the surroundings are environmental factors to be considered.

Assessment of effects of individual parameters described above, which is one of the subjects to be solved yet, is beyond the scope of this report. Instead, this section will consider the effects of ballast properties on track performance, and the development of ballast properties through mechanical processes such as tamping, traffic and compaction.

Track Stability Considerations. One of the ways that ballast affects track performance is through its effect on track stability including lateral, longitudinal, and vertical tie resistance, track settlement, and track geometry changes. The effects have been mainly measured in terms of such physical parameters. Hence, in this section, the effects of ballast will be described in those terms.

In an attempt to establish correlations between various laboratory-determined index properties and performance in the field, CNR placed ten different ballasts in adjoining 400-m sections of track and annually observed track and ballast characteristics as well as made classification tests on ballast samples taken from each section (Ref. 54). Each section was evaluated by eye in terms of the amount of ballast breakdown, stability of the ballast bed, and amount of maintenance required.

Raymond (Ref. 42, 241, 242) conceived excessive breakdown of the ballast particles and instability of the ballast bed as the two major reasons for unsatisfactory track performance at the test track. Thus he assigned two different field ratings to each ballast, one for breakdown and the other for stability. The laboratory tests conducted to represent ballast breakdown characteristics measured specific gravity, absorption, LA abrasion resistance, chemical soundness, crushing value, and freeze-thaw resistance. Tests to define the stability characteristics measured flakiness index, sphericity, roundness, and elongation index. These laboratory test results were then statistically correlated to the field ratings. No apparent correlation was noticed except with the sodium sulphate soundness value. The stability field rating showed poor correlation with the shape tests, although sphericity was the best among the tests. The fact that the index tests do not consider or simulate actual field conditions at all, and that the field rating was rather subjective, may explain such poor correlation.

Even though a satisfactory correlation between the index properties and track performance has not been established, the various specifications for ballast from different railroads include and are based primarily upon such tests as part of the criteria determining acceptance of ballast, largely based on some consensus of practical experience. For example, Conrail includes absorption, Deval abrasion, LA abrasion, sodium sulphate soundness, and cementing value, along with gradation requirements. Certainly more research on this topic is needed.

Ballast Compaction Benefits. Ballast compaction has been recognized as one of the important factors influencing track performance. For example, Hardy (Ref. 243) noted from a series of field tests conducted on a Canadian National branch line, that the uniformity of the ballast compaction appeared to be of greater importance than the thickness of ballast with respect to rail stresses, which he considered as an indicator of the track performance. Based on the measurement of the maximum rail stresses in the web and flange of 60-lb and 100-lb rails for the various ballast and subgrade conditions under different loading, Hardy emphasized the need of ballast compaction under the ties to reduce effects of the irregularities in the ballast and, therefore, to increase the stability of the track.

However, assessment of effects of ballast compaction on track performance is very difficult. Because of the large number of parameters involved, and the lack of methods for determining ballast compaction, the current understanding of ballast compaction and its effects on various aspects of track performance are very limited. Numerous factors have been identified that affect ballast densification during construction, maintenance, and service, as summarized in Table 7.2. Most of the factors represent specific field conditions, and their relative importance varies significantly with time and environmental changes,

Table 7.2. Factors Influencing Ballast Densification during Construction, Maintenance and Service

1. Subgrade	. Ballast supporting conditions . Drainage . Gradation
2. Ballast	. Type . Size, shape, and gradation . Durability and hardness . Ballast layer dimensions . Presence of subballast . Degree of fouling
3. Track Structure	. Type, size, and conditions of rail, fastening, and ties . Geometry
4. Maintenance Procedures	. Quality and amount of lifting, lining, tamping, and compaction . Frequency . Sequence of different maintenance operations
5. Compaction Procedures	. Compactor design characteristics and operation, such as tool arrangement, shape of compacting shoe, static pressure, dynamic pressure, frequency of vibration, duration of vibration, etc.
6. Traffic	. Density . Speed . Axle load
7. Environment	. Precipitation . Temperature, including frost effects

as well as with location. Therefore, the problem of ballast compaction must be examined taking into consideration all the parameters related to each track component, methods and quality of maintenance work, existing conditions prior to the maintenance and compaction work, and traffic conditions.

When a track deteriorates beyond acceptable limits, the track is subjected to various maintenance operations, such as skin lifting, ballasting, and tamping. The tamping operation, employing vibrating and squeezing tools, inevitably disturbs the ballast which had been previously compacted by traffic. The surfacing and lining operation changes the pattern of tie support appreciably. Voids are created under the center of the tie. Under the tie in the tamped zone squeezing creates a short tapered column or pyramid of perhaps moderately compacted ballast beneath the tie with loose and disturbed ballast on either side. This short column of compacted ballast with fairly loose surrounding ballast in the crib area would reduce track stability and create rapid settlement of the track under subsequent traffic, particularly in the first few loads following tamping. Raising and tamping also lowers the level of the ballast in the crib on the sides of the tie in the vicinity of the rail and therefore reduces the resistance to lateral or longitudinal displacement. Such reductions may cause a seriously unstable track condition after tamping, particularly in a case of continuously welded rail at high ambient temperatures.

A range of 30 to 70 percent reduction in lateral and longitudinal resistance of track after tamping compared to undisturbed track has been reported depending on the track conditions, methods of measurement, and the nature of the maintenance work involved. However, as will be seen later, this reduction will be slowly recovered with the accumulation of further traffic. For example, it has been generally reported that about 1,000,000 gross tons of traffic are required for full recovery of the lost resistance.

To reduce this adverse effect of tamping, and to enhance the process of restoration of track stability, ballast crib compaction has been suggested. By adding ballast, and compacting the loose ballast in the crib and on the shoulder immediately after tamping, the stability deficiencies caused by ballast tamping and lining are reported to be offset at least partially. The settlement of track under traffic, as well as differential settlement, may be limited or avoided, and the lateral and longitudinal resistance can be assured even immediately after maintenance work.

Ballast compaction immediately after tamping is reported to reduce 1) the cost of slow orders that usually are put into effect while traffic recompacts disturbed ballast, 2) the possibility of sun kinks that may occur in some areas after track maintenance during hot weather, 3) the possibility of damage to subgrade of marginal strength that may occur while pedestals of ballast tamped under the tie are transmitting concentrated loads, but particularly the reduction in maintenance frequency, have not been clearly proven in research to date.

Ballast crib compaction has received a significant amount of attention during the last two decades. Various railroads in European countries had an earlier start on this approach, than in the U.S. and Canada, because of the much tighter track tolerances specified in Europe. Even though a wide range of results of measurements on track have been reported, ballast crib compaction shows fairly consistent patterns. These are examined in the following sections of the report based on measurements of physical state, lateral tie resistance, longitudinal tie resistance, rail settlement and track geometry changes.

7.4 CRIB COMPACTION STUDIES

Bearing Resistance and Density. The Canadian National Railways (Ref. 137) investigated the effects of various track maintenance operations, including ballast compaction, on ballast condition expressed in terms of a ballast bearing resistance from a plate load test. The ballast bearing index was defined as the vertical force per unit area under a 5 -in.-diameter plate on the ballast surface at 0.3-in. deflection. This parameter was measured in four different positions around and under the ties after track lining, surfacing, and crib compaction.

Even though significant variations in track conditions were noticed, the results (Table 7.3) yielded several interesting facts. The track tamping, surfacing, and lining work disturbed the track substantially. Crib and shoulder compaction after track work appears to restore the ballast state to a degree that varies significantly, depending on the type of track work and the position of the ballast. Generally, in the shoulders and crib area where the compaction is applied, the bearing resistance is increased to a value equal to or greater than that prior to tamping. However, under the tie the effect of compaction was very limited.

Birmann and Cabos (Ref. 203) measured the effect of ballast compaction directly by determining changes in density. In a series of field tests at a northern German track site, the ballast density was measured using a nuclear method. Data were obtained in the same locations before and after traffic under different compactor static pressure, dynamic force, vibration frequency, and vibration period. The results indicated the following:

1. An increase in the static compaction pressure from 125 to 375 kg had a significant influence on ballast density (Fig. 7.7).

Table 7.3. Variations of Ballast Bearing Index with Various Maintenance Operations (Ref. 137)

	Measurement	Shoulder at Surface	Crib at Surface	Crib at Depth	Under Tie
1. Original	Original BBI Values (lb/sq in. at 0.3 in.)	131	264	260	535
2. Compaction	Percent of Original Values				
Shoulder Only		103	115	113	110
Crib & Shoulder		106	121	134	96
3. Track Work Only	"				
Lining Only		50	73	102	100
Surfacing & Lining		73	24	70	27
Undercutting Only		51	56	71	36
4. Track Work and Compaction	"				
Lining & Shoulder		74	86	138	136
Lining, Shoulder & Crib		117	86	105	99
Surfacing & Shoulder		96	34	63	46
Surfacing, Shoulder & Crib Compaction		158	116	114	73
Undercutting & Shoulder		88	58	92	57
Undercutting, Shoulder & Crib Compaction		107	126	125	57

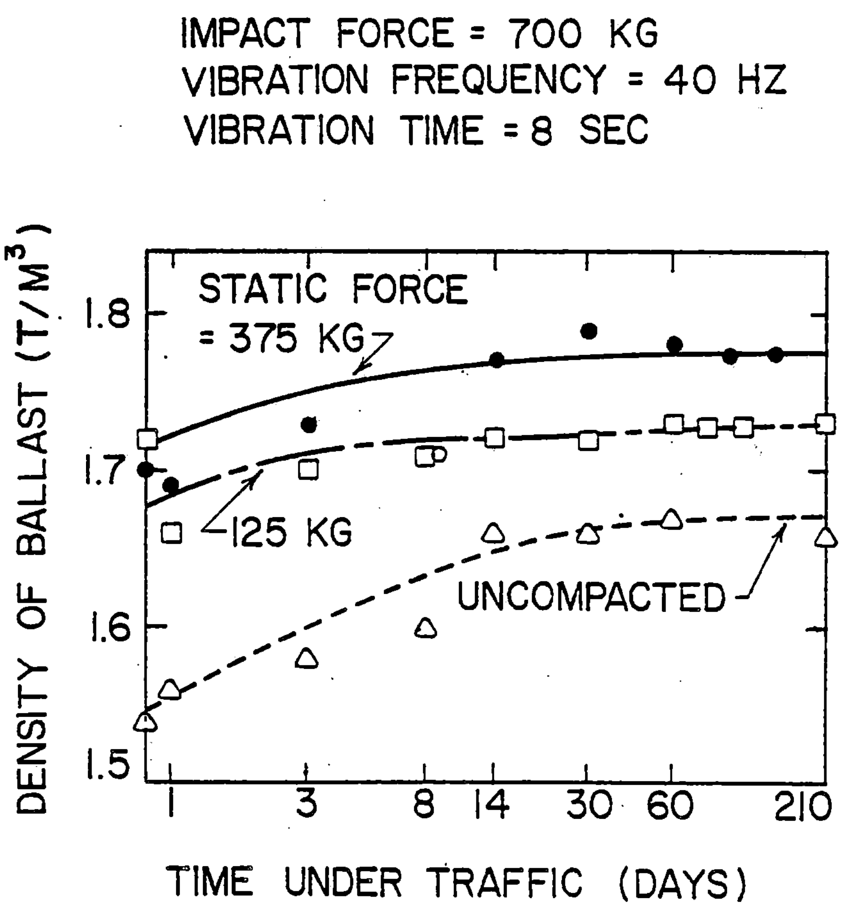


Figure 7.7. Effect of Static Force on Compaction of Ballast with Windhoff Machine of German Federal Railroad (Ref. 203)

2. In contrast, an increase in the dynamic force from 700 to 1000 kg appeared to have a less important effect (Fig. 7.8). However, the method of measuring the dynamic force was not indicated, so these values may not be correct.

3. An increase of vibration period from 10 to 12 seconds resulted in higher densities (Fig. 7.9).

4. A frequency change from 40 to 47 cps had no appreciable influence.

5. An increase in the ballast density with accummulation of traffic loads was apparent in all of the cases. However, the change in density due to traffic was considerably smaller in the compacted sections than in the uncompacted sections.

6. After seven months under a daily traffic of 23,000 tons, all the compacted sections still had a higher ballast density than the uncompacted sections.

The Yugoslav Railroad (Ref. 244) also reported nuclear density measurements before and after crib compaction with the Plasser crib compactor VDM800. Density measurements were made at 20 and 30 cm below the top of the tie, at 12 locations each. How the ballast density was measured is not known. The results consistently showed higher ballast density after compaction than before compaction (Fig. 7.10).

<u>Lateral Resistance</u>. Eisenmann and Gnad (Refs. 245,246) conducted lateral tie tests using panels of 7 ties at a German Federal Railway (DB) track. They reported that the lateral resistance was greatest when the track had settled under traffic. Tamping combined with a raise of 2 to 3 cm reduced the lateral resistance to approximately 30% of the original, pre-tamped value. However, when the crib and shoulder compactor was used immediately after tamping,

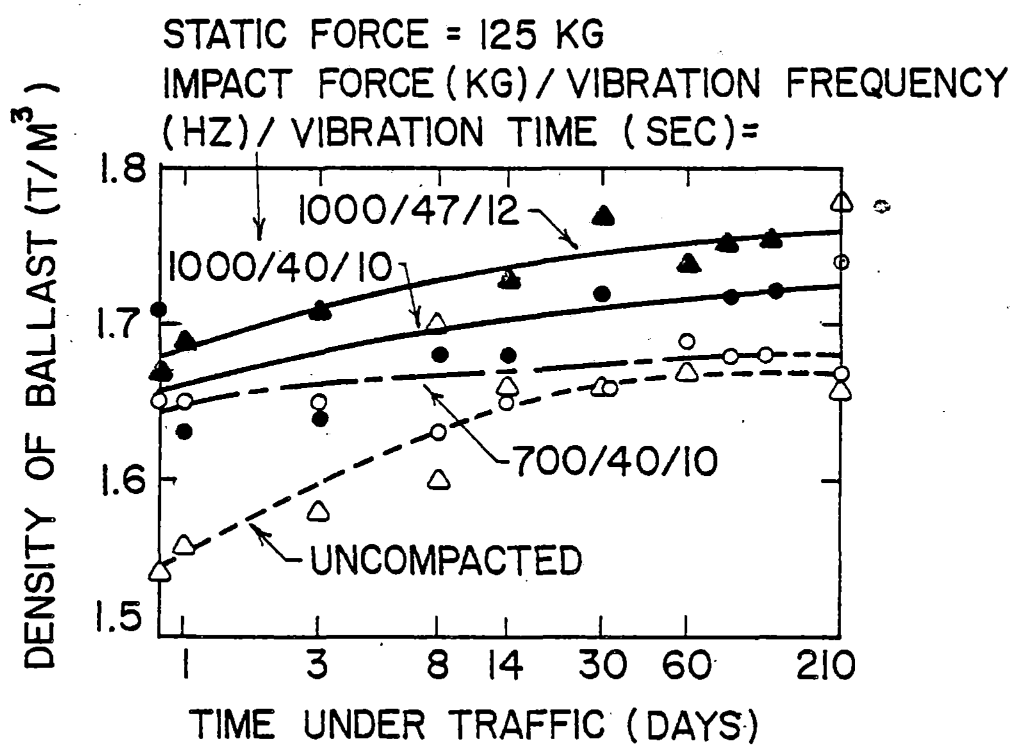


Figure 7.8. Effect of Impact Force and Vibration Conditions on Compaction of Ballast with Windhoff Machine of German Federal Railroad (Ref. 203)

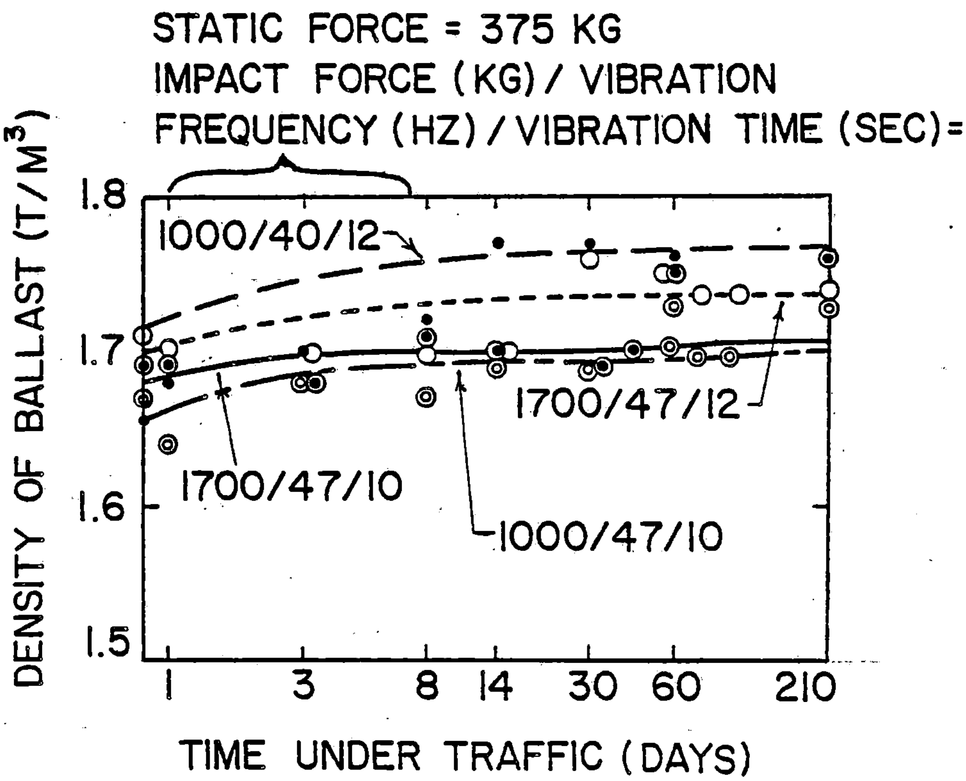


Figure 7.9. Further Examples of Effect of Impact Force and Vibration Conditions of Compaction of Ballast with Windhoff Machine of German Federal Railroad (Ref. 203)

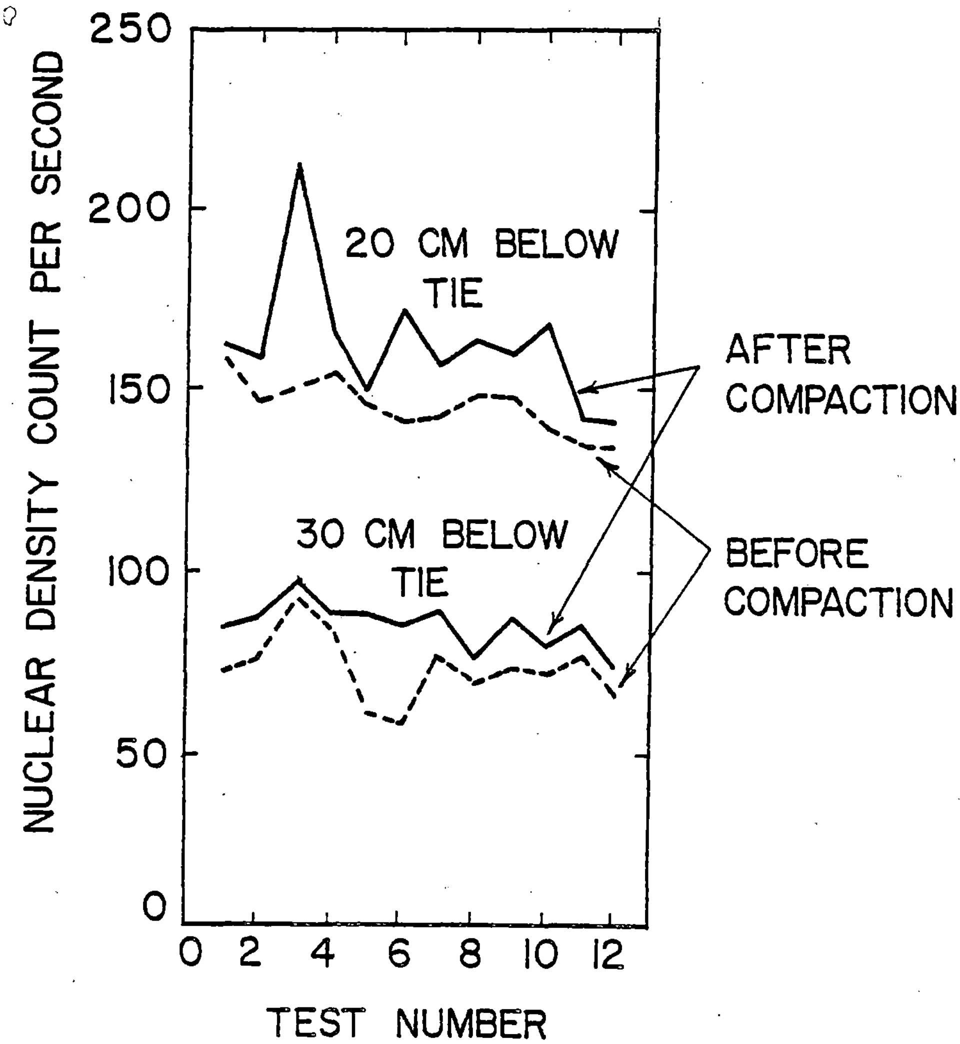


Figure 7.10. Ballast Density Increase with Ballast Compaction (Ref. 244)

the lateral resistance was increased prior to traffic to about 60% of the initial value, but the original, pre-tamped resistance was not reached until a traffic load of some 250,000 tons had been accumulated. An increase of 30% in lateral resistance by crib and shoulder compaction was reported to equivalent to a traffic load of approximately 60,000 tons. The authors also noted that when the ballast compaction followed behind the tamper without intervening traffic, not only was the resistance significantly increased, but the uniformity improved as well.

Birmann and Cabos (Ref. 203) also conducted field tests to investigate the compaction effects of a European crib ballast compactor. A tangent section of test track consisting of concrete ties on uniform subgrade in northern Germany was subjected to tamping. The effect of compaction was measured periodically as a function of track service time and traffic.

The lateral resistance test results before and after compaction are compared in Fig. 7.11. An increase of lateral resistance at 2 mm displacement from 1370 to 1520 kg/m per unit track length was observed (i.e., 10.2% increase). A static load of 6 tons reportedly was distributed over two axles 3.0 m apart during tests. However, it is not clear how the tests were conducted.

Plasser and Theurer (Ref. 247) performed lateral tie push tests to evaluate the capability of their tamping machine and ballast compactor on a 900 m straight concrete tie track designed for 160 km/hr speed. The track was divided into 4 sections prepared as follows: 1) tamped once, 2) tamped and crib compacted, 3) tamped twice, and 4) tamped twice and crib compacted. In each section, 20 ties were tested in individual tie push tests before and after the above maintenance and after 45,000 tons and 90,000 tons of traffic. After tamping, a ballast plow filled the cribs. Compaction was then performed using

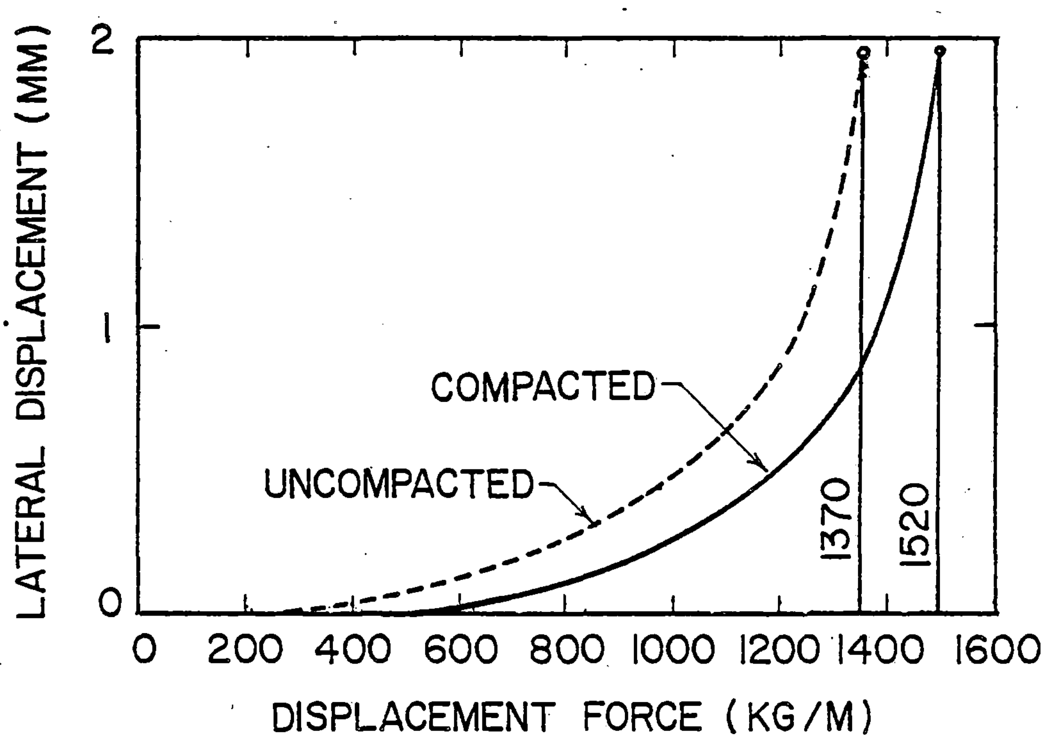


Figure 7.11. Resistance per Unit Length to Lateral Displacement of B58A Concrete Sleeper Track with Static Load of Six Tons Distributed over Two Axles 3 m Apart (Ref. 203)

a Plasser VDM800-U machine. The maintenance including ballast compaction accumulated an equivalent traffic load of 10,000 tons.

Fig. 7.12 summarizes the test results taken immediately after the maintenance work. The lateral resistance at 1 mm and 2 mm displacements dropped off to 47% of the values for the original undisturbed track. Crib compaction restored the resistance to 57%, i.e., about 10% increase. No difference was noticed between single and double tamping. The loss of lateral resistance was recovered to 61% of the undisturbed values after 45,000 tons of traffic accumulation (Fig. 7.13), and to about 65% after 90,000 tons (Fig. 7.14).

Klugar (Ref. 248) also reported similar test results obtained from 32 tests conducted on a Vienna-Gnuend line in Austria, where 4.7 m track panels of 7 ties each were tested. The track maintenance performed included lifting, lining, surfacing, crib compaction, and shoulder compaction. Figure 7.15 summarizes the results, showing the lateral resistance differences between various maintenance procedures. Curve 1 shows the resistance for the undisturbed track. This is the highest resistance measured. The lowest resistance, curve 4, was obtained for lifting and tamping with no compaction. In this case the resistance was 79% of the undisturbed value at 2 mm displacement. Adding crib compaction, curve 3, increased the resistance to 86% of the undisturbed case; and adding both crib and shoulder compaction, curve 2, increased the resistance to 90% of the undisturbed case. Thus, shoulder compaction had only a small influence on lateral resistance.

The total resistance was assumed to consist of 1) the friction caused by the ballast pressure against the tie sides, 2) the passive resistance of ballast against the end of the tie, and 3) the resistance along the tie bottom. Klugar estimated the contribution of the crib and shoulder to the total

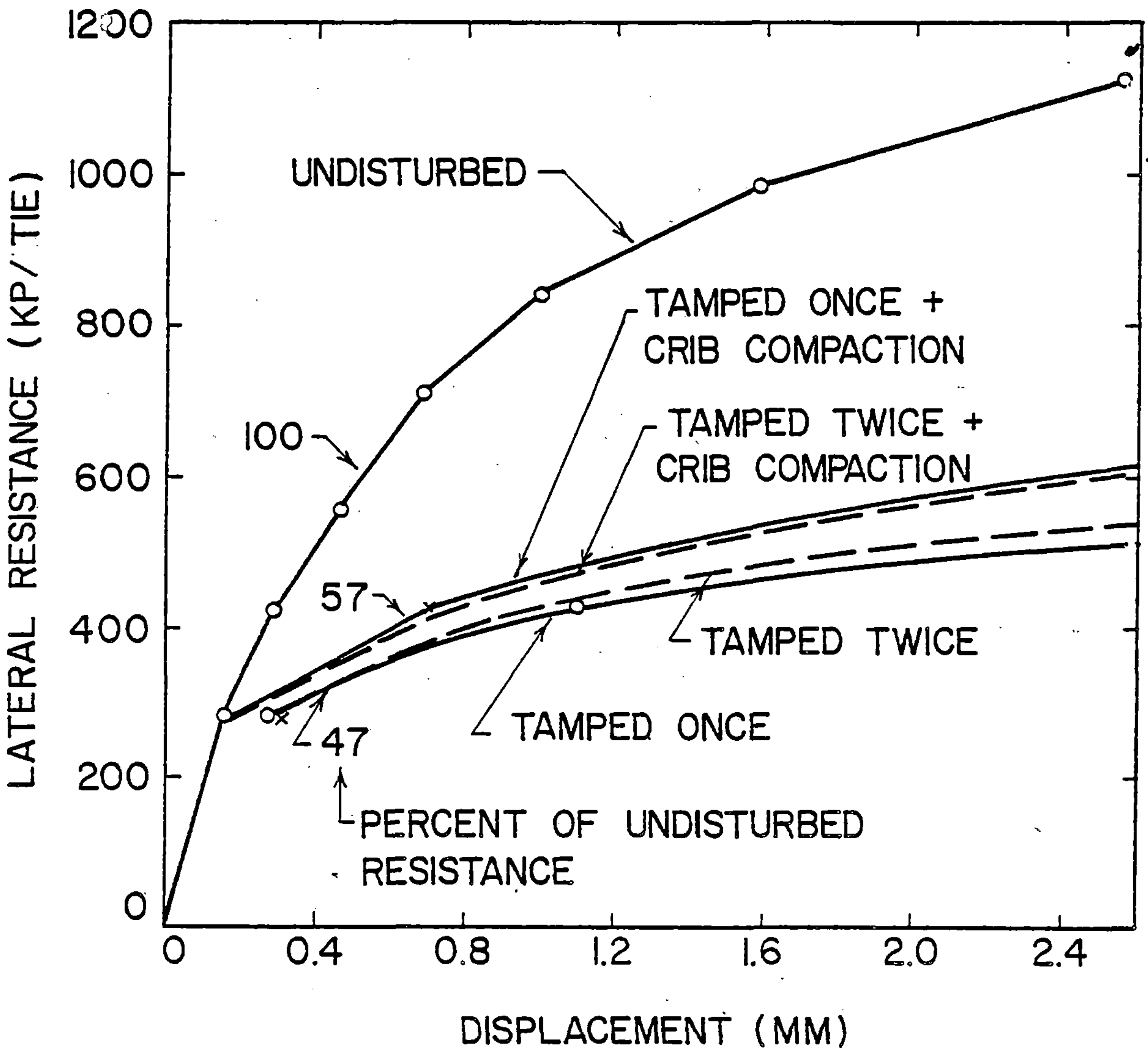


Figure 7.12. Variations of Lateral Resistance of Single Ties with Different Maintenance Operations After 10,000 Ton Traffic (Ref. 247)

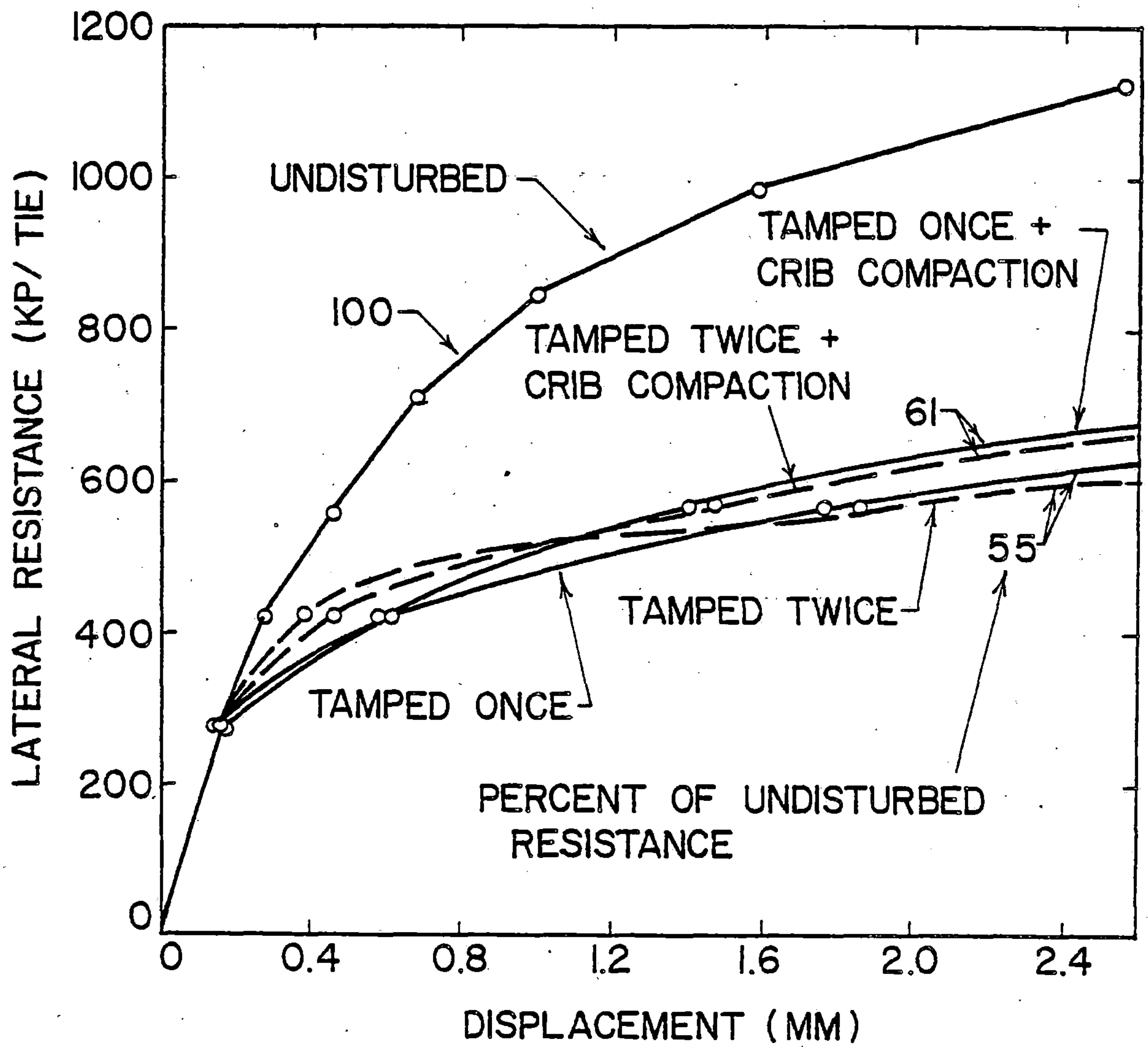


Figure 7.13. Variations of Lateral Resistance of Single Ties with Different Maintenance Operations After 45,000 Ton Traffic (Ref. 247)

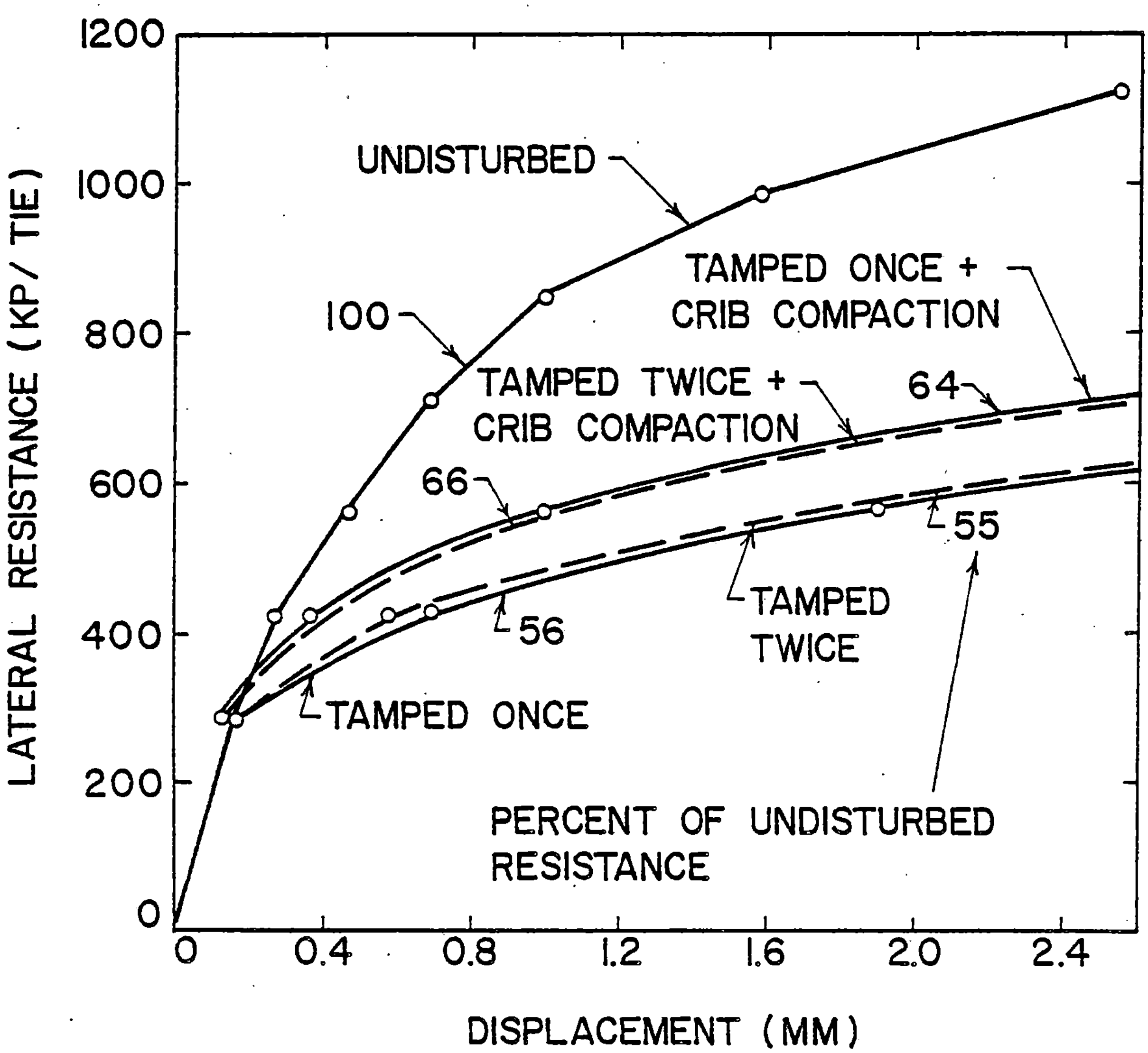


Figure 7.14. Variations of Lateral Resistance of Single Ties with Different Maintenance Operations After 90,000 Ton Traffic (Ref. 247)

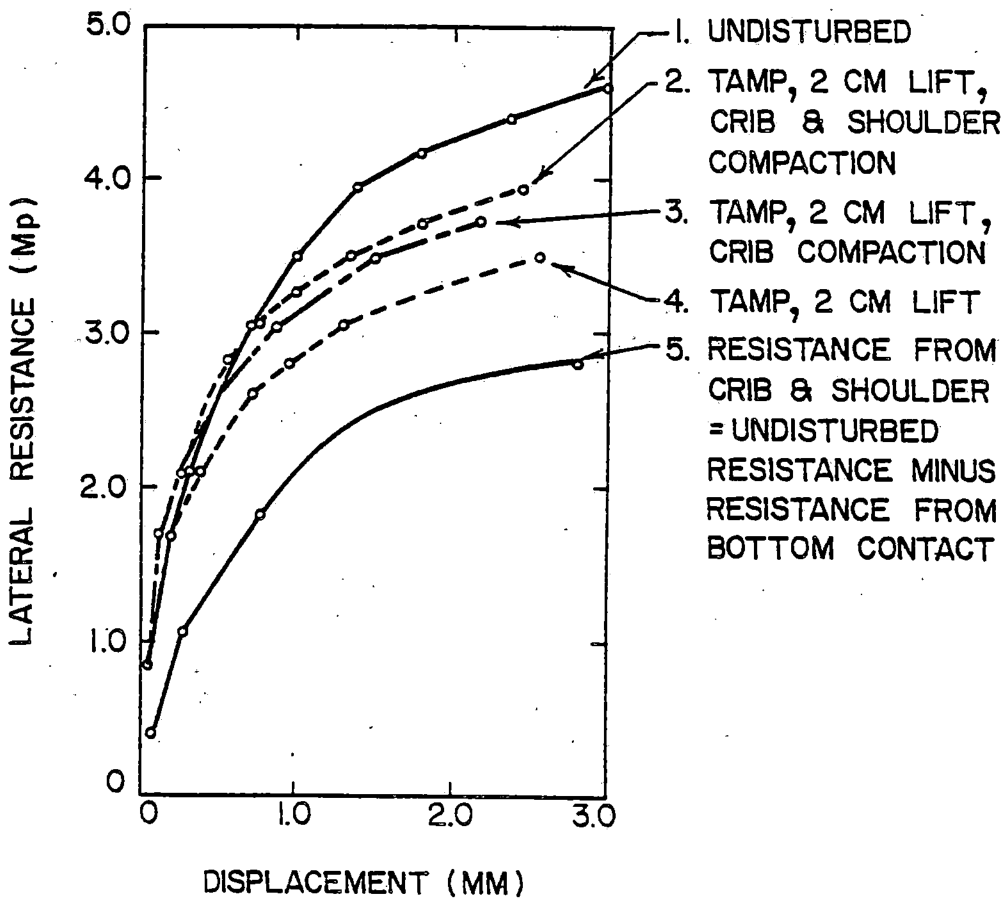


Figure 7.15. Comparison of Lateral Resistance of Seven-Tie Panel for Different Maintenance Operations, Zero Lift (Ref. 248)

resistance in the undisturbed state, U, by subtracting the calculated value of resistance for the first two of these components (author's U_2) from the total resistance, U. This resulting curve 5 represents 63% of the undisturbed resistance (curve 1) and about 70% of the resistance after tamping and compacting (curve 2).

Another interesting aspect of Klugar's study was the investigation of the effect of the amount of lift on the lateral displacement resistance. The results, shown in Fig. 7.16, are based on data in Ref. 249. For all three maintenance procedures (curves 2, 3, and 4), as the amount of raise or lift increased, the lateral resistance decreased.

Matisa Materiel Industriel, SA, (Ref. 250) also investigated the effectiveness of its crib and shoulder compactor. A series of field tests involved with a wood tie track at the German Railways examined the lateral tie resistance of a track panel (7 ties) for different maintenance procedures, such as lifting, tamping, and leveling, with combinations of crib and/or shoulder compaction. The vibration period during compaction was also varied from 5 to 9 seconds. The results indicated that with crib and shoulder compaction, the average lateral resistance of single ties reached 90% of the initial, pre-tamped values, compared to about 40% immediately after tamping. Shoulder compaction alone increased the resistance to approximately 60%.

In another series of single tie lateral tie resistance tests at an Italian railroad between Milan and Bologna, Matisa (Ref. 250) observed an increase of 39% in the average lateral resistance increase after ballast compaction. However, it was noted that concrete tie resistance was much higher at all times, about 2.6 times the resistance of wood ties.

Powell (Ref. 228) also investigated the effectiveness of a ballast

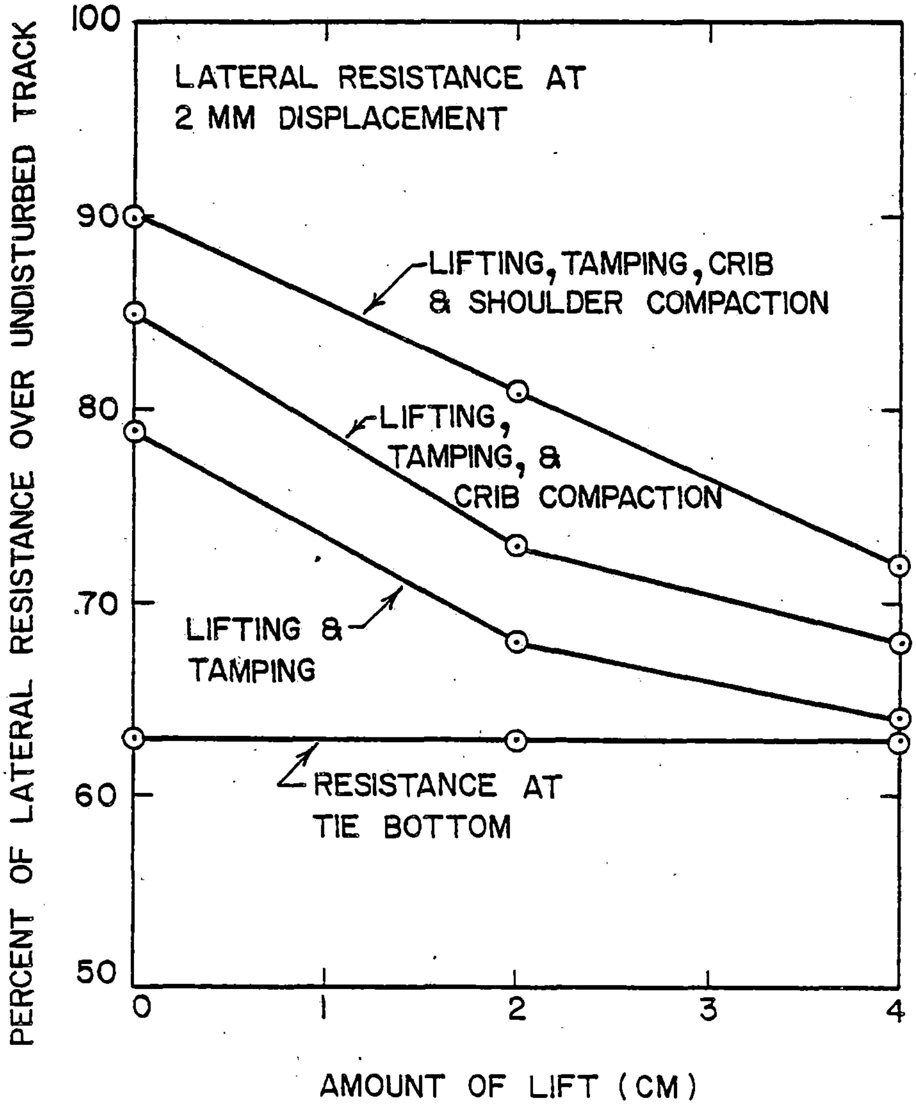


Figure 7.16. Effects of Track Raise During Maintenance (Ref. 249)

compactor (Plasser VDM800) in terms of lateral resistance of single ties. The track, consisting of CS3 concrete ties on limestone ballast, was tamped and compacted, with either 1 or 2 in. of ballast overfill above the top of ties. The amount of track lift during tamping was only 1 cm. The results indicated that any form of compaction considerably improved the value of lateral ballast resistance from the loose condition after tamping. Interestingly, it was noted that crib compaction reduced the resistance value over tamped only track when there was not sufficient ballast. However, when plenty of ballast was present before compaction, a considerable improvement of lateral resistance over tamped track was found.

Riessberger (Ref. 251) reviewed various studies on the subject in different European countries, including some of the above described studies, and concluded that ballast compaction yields a substantial improvement in track geometry and in the lateral stability of track along with the vertical and longitudinal stabilities. He also concluded that ballast compaction increases not only the average resistance but also the uniformity. Fig. 7.17, which summarizes the review indicates that the lateral resistance for the cases in which compaction was used was about 50 to 65% of the resistance of the undisturbed track.

The Federal Railroad Administration of US/DOT acquired a ballast compactor (Plasser VMD800) in 1973 and initiated a very extensive series of field tests to study the effectiveness of crib and shoulder ballast compaction (Ref. 205). With the participation of five different railroads (Southern Railways, Boston and Maine, Penn Central, St. Louis and Southwestern, and Missouri Pacific) data were collected to examine the effectiveness of using the compactor in conjunction with the tamping operation on in-service lines. For comparison, data were also collected on the selected test lines with the same level of maintenance but

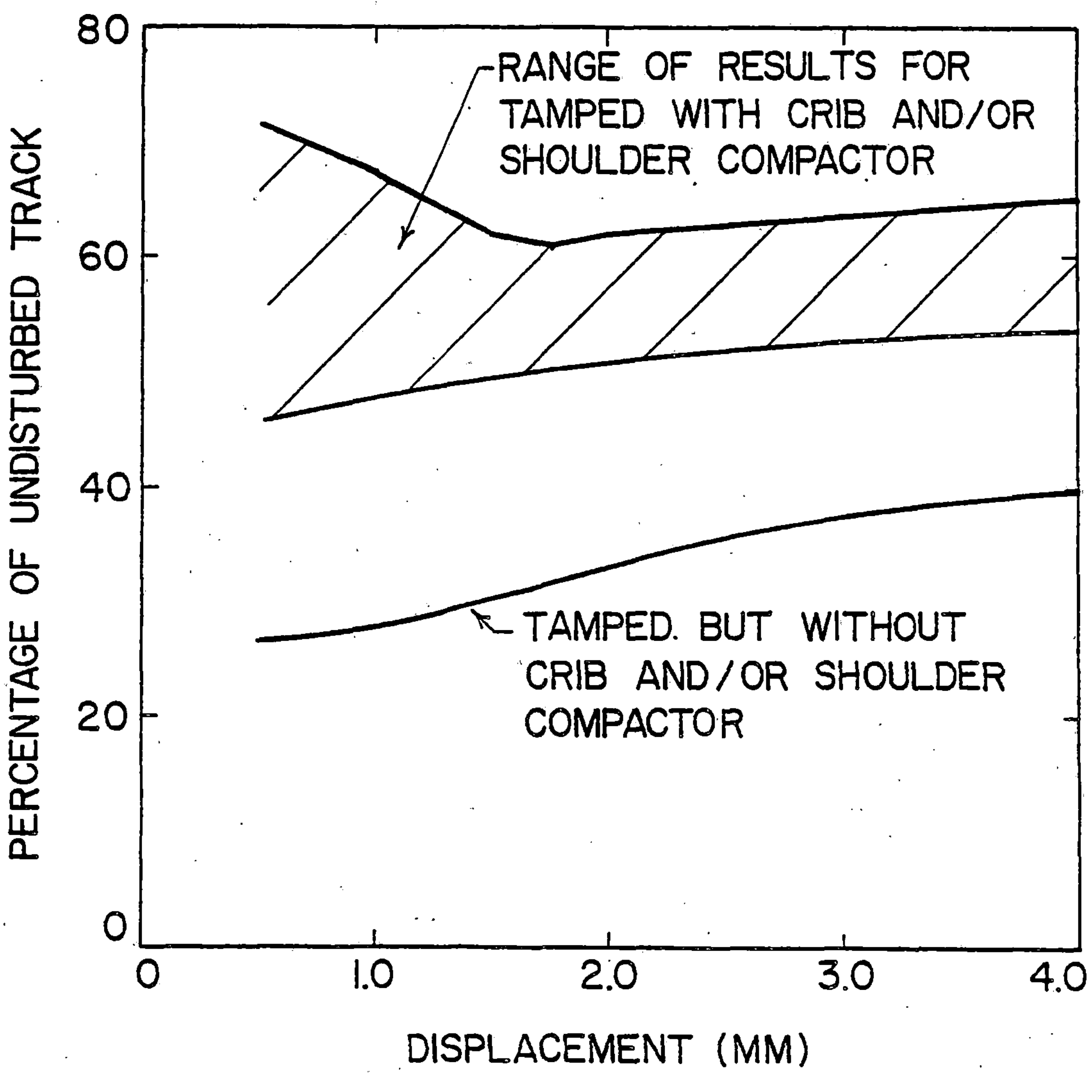


Figure 7.17. Lateral Track Resistance with Various Track Maintenance Machines as a Percent of Resistance for Undisturbed Track (Ref. 202)

without ballast compaction. The track conditions and traffic level of each of the participating railroads were varied, but all of them had wood ties on crushed AREA No. 4 granite ballast, except that limestone ballast of the same size was used at the Penn Central site.

Each individual railroad selected four test sections, two in the body of a curve and two on tangent track. Each section was approximately 1/8 mile in length. All four sections were subjected to the same level of maintenance, but only one section on each of the curved and tangent tracks was compacted using the FRA compactor. The measurements made included track settlement surveys, tie-displacement tests, track modulus tests, joint profile tests, and a track geometry survey. These were taken immediately after the specified maintenance work, and before any traffic, as well as at 0.5, 2.0, 5.0 and 10.0 MGT of traffic.

Westin (Ref. 252) reported the test results obtained from sections of tangent track on the B & M and the Southern railroads. In these particular cases, lateral tie push tests were conducted on 12 to 20 single ties in both uncompacted and compacted sections at various time intervals. The results showed that ballast compaction raised the initial tie resistance from 60% after tamping to 70% of its final expected value after compaction. However, it was indicated that the compactor had no long-term effect on lateral tie resistance, and there was little difference between compacted and uncompacted ties after 1/2 MGT. The improvement in lateral tie resistance from the use of the compactor was concluded to be no more than that achieved by an equivalent traffic load of 100,000 to 200,000 gross tons.

Ensco reported (Ref. 236) the detailed test conditions, procedures, and results of all the FRA testing. Even though there were variations in the test results obtained depending on the difference in procedures used, such as type of tests (individual vs panel), configuration and conditions of ballast layer, rails, tie, and fasteners, the following general conclusions were reached:

1. Compaction increased lateral resistance by an average of approximately 40% of the values after tamping.

2. Compaction produced the least improvement in lateral resistance with new concrete ties, approximately 17% increase over the tamped track, and produced the greatest improvement in wood ties, approximately 55% increase.

3. Compaction after tamping produced an average lateral stabilizing effect equivalent to approximately 440,000 gross tons of traffic when measured at 4 mm displacement.

The Canadian National Railways (Ref.253) also conducted a series of field tests to determine the effectiveness of the Plasser Compactor CPM-800-R. The test site consisted of 132.lb RE rail, fully tie-plate, 6 in. x 8 in. wood ties with some 7 in. x 9 in. ties, and crushed rock ballast. The lateral resistance of single ties was measured after tamping, after compaction, and after 0.3, 0.6, and 1.2 MGT of traffic. A total of 20 groups of ties were tested after tamping only, and after tamping and compaction. Each group consisted of six individual ties. The results indicated the following:

1. The lateral resistance after tamping and before any traffic load passed over the site was 55% of the previously undisturbed values.

2. The lateral resistance after tamping and compacting and before any load passed over the site was 85% of the previously undisturbed value.

3. After 0.6 MGT of traffic had passed over the site, the percentages were raised to 108 for tamped only and 116 for tamped and compacted sections. (The reason that the percentage figures are greater than 100% has never been explained).

4. After 1.2 MGT of traffic the percentages were 143 for tamped only and 152 for tamped and compacted sections.

5. A comparison of ties of different sizes indicated that a 7 in. x 9 in. tie has a resistance to initial displacement that is 33% greater than a 6 in. x 8 in. tie.

Based on these test results, the CNR decided not to place a slow order on the finished track work when compaction was used, and trains were reportedly permitted to go at timetable speed immediately after compaction.

Track Settlement. There are various reasons for track settlement, such as 1) volume reduction from particle rearrangement and from degradation of ballast during maintenance and under traffic, 2) ballast sinking and/or pumping, 3) ballast particle lateral displacement, and 4) subgrade consolidation. Therefore, measurement of track settlement as a means for evaluating the effectiveness of ballast compaction has serious limitations. However, various studies have reported that ballast compaction reduces the settlement of track after surfacing or tie replacement, therefore enhancing the ability of the track to achieve a stable surface in a reduced time, and also increases the uniformity of the settlement.

As shown in Fig. 7.18, obtained from an ORE study (Ref. 254), the tamped track settled at a decreasing rate for an indefinite period. On the compacted track, although initial settlement was greater, that occurring after 100,000 tons of traffic was negligible. However, the amount of data is too limited to insure valid conclusions.

A different pattern, but also in favor of ballast compaction, is shown in Fig. 7.19 (Ref. 254). A load moved repeatedly back and forth over a test track producing the settlement shown, which increases with number of load cycles.

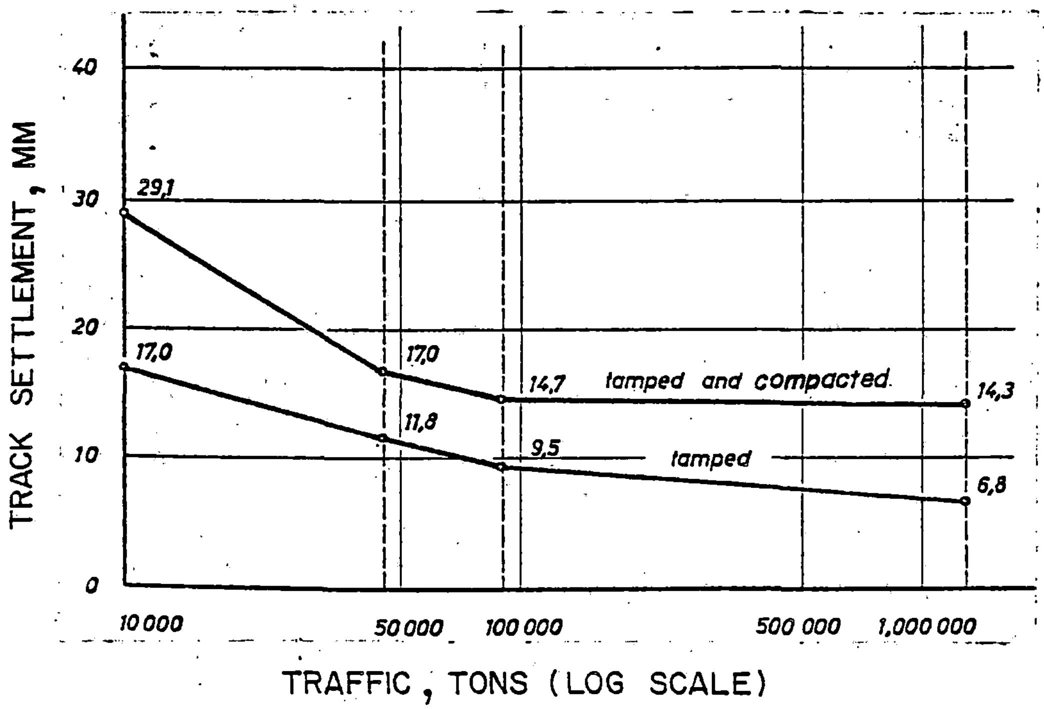


Figure 7.18. Effect of Ballast Compaction on Track Settlement with Traffic (Ref. 254)

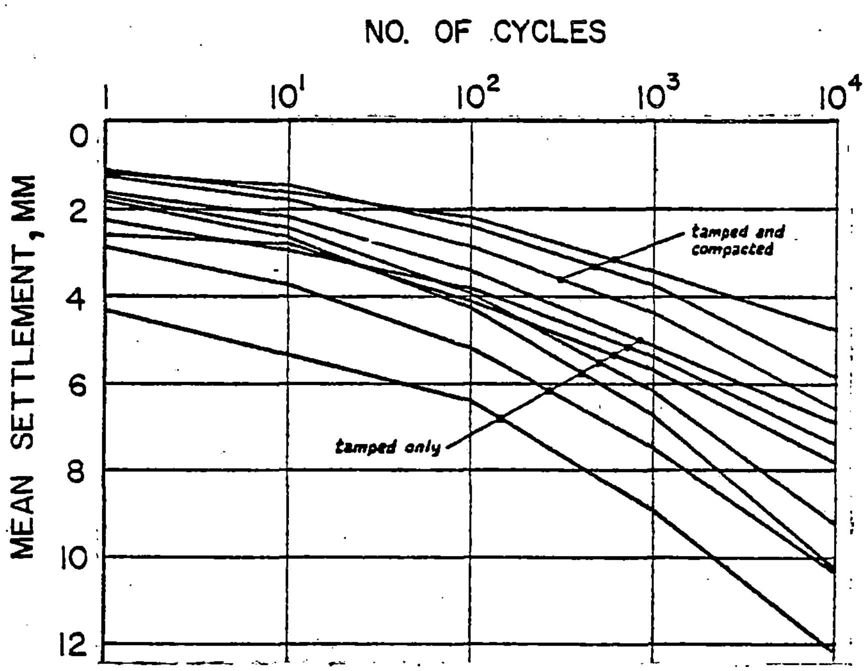


Figure 7.19. Effect of Accumulating Load Cycles on Settlement of Test Track in Britain, With and Without Crib Consolidation (Ref. 254)

Compaction reduced the overall average settlement after a given number of cycles to about 60% of that for tamped only track. In the same test, the increase in uniformity of the ballast layer gained by compaction was represented by the standard deviation of settlement readings for each ballast condition and load repetition. The values of standard deviation for the tamped-only track were 2 to 4 times larger than those for the tamped and compacted track.

Similar results to the above-mentioned studies have been obtained in the FRA field testing programs described previously (Fig. 7.20 and 7.21). Compaction reduced the settlement of ties at rail joints caused by 155,000 gross tons of traffic by approximately 20% in curved track and 15% in tangent track.

Track Geometry. Birmann and Cabos (Ref. 203) investigated the effects of compaction on track geometry deterioration during the field tests of compactor evaluation described earlier. The track geometry changes were determined during the track acceptance survey. Fig. 7.22 shows the frequency curves of the number of points where the relative level had remained constant and where it had either deteriorated or improved by 1 mm, 2 mm, or 3 mm. As can be seen in the figure, in the uncompacted sections, the relative levels have consistently deteriorated, and the number of major faults in the level has increased.

Track quality measurements were taken with a special recording car on a mainline of the Yugoslav Railways (Ref. 244) for a few months before and a year after a track maintenance operation including ballast compaction. Figure 7.23 illustrates the rate of track deterioration in terms of twist, in the period after tamping compared to the period after tamping and compaction.

Apart from a period of 24 days (4/21/70 to 5/14/70) immediately following the operation, the rate of deterioration of track quality with compaction was about 50% of that previously recorded without compaction. A 30% increase in

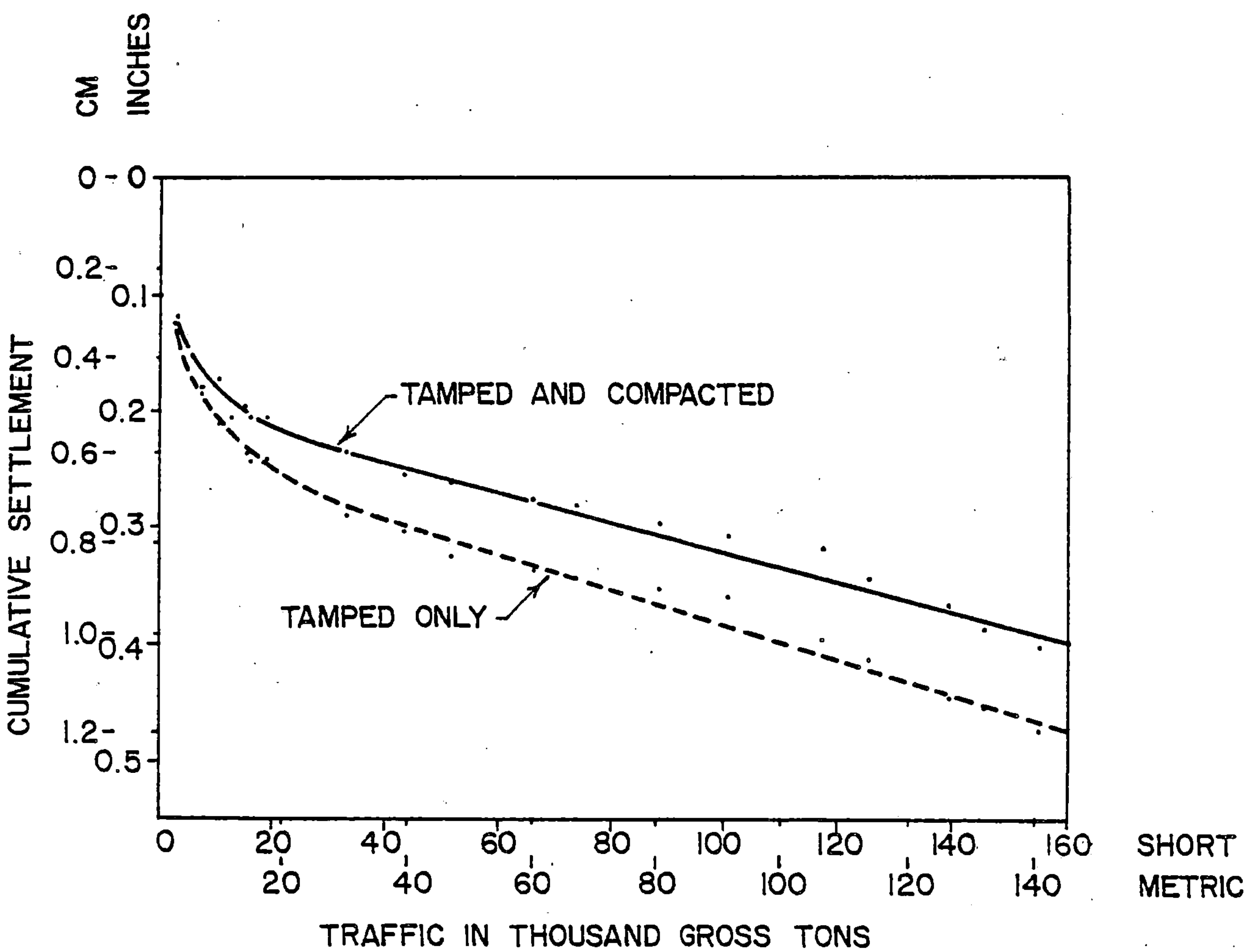


Figure 7.20. Settlement Versus Traffic at Bolted Joints in Tangent Track (Ref. 236)

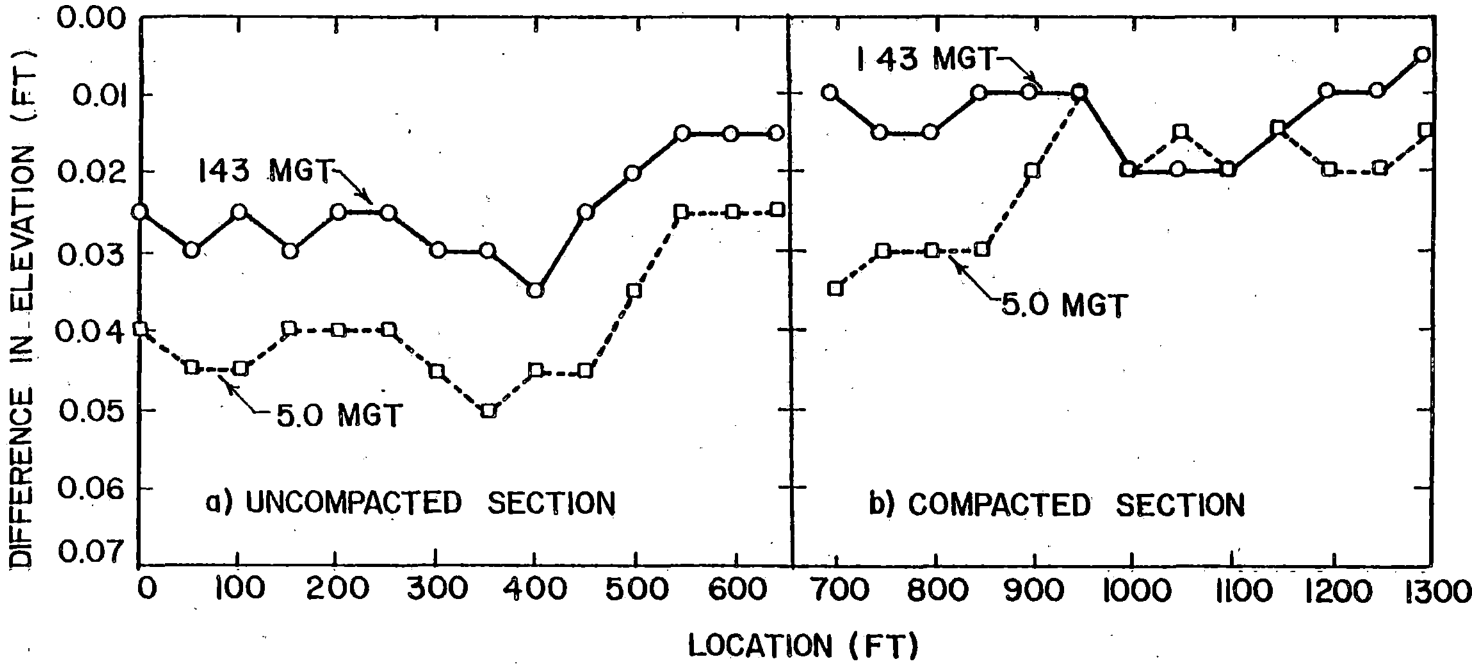


Figure 7.21. Comparison of Track Settlements between Compacted and Uncompacted Sections, Tangent Track (Ref. 205)

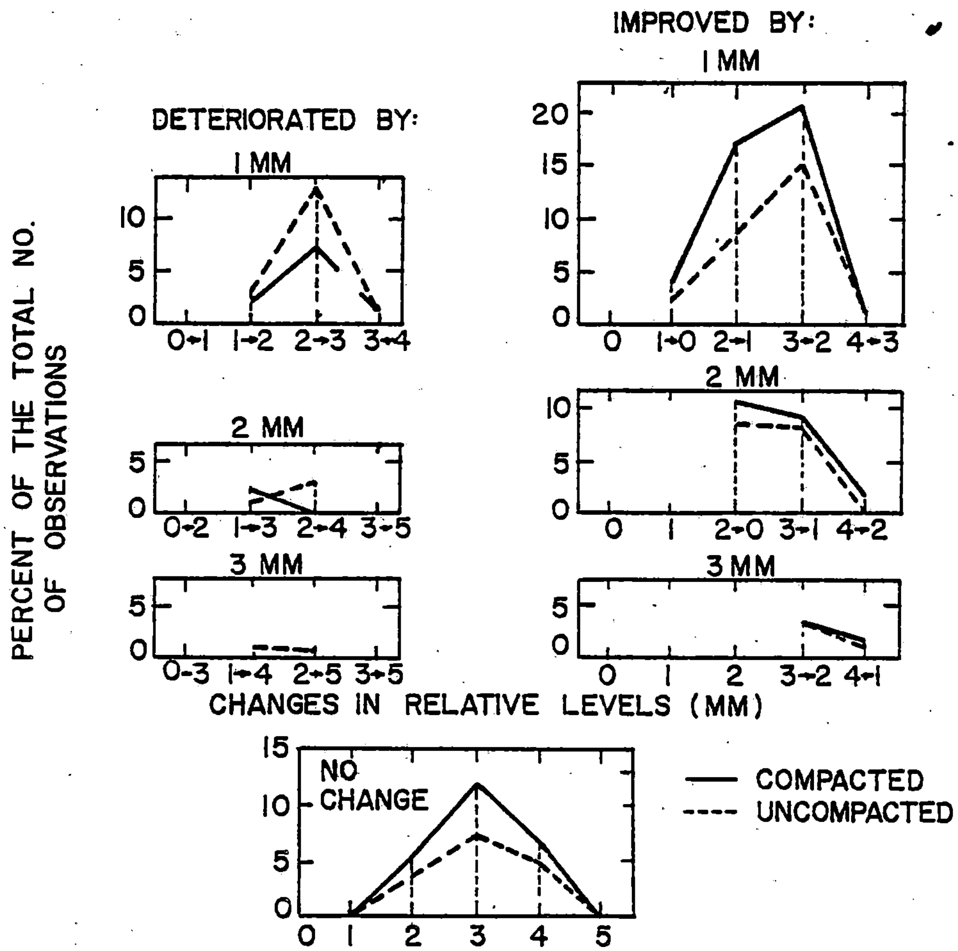


Figure 7.22. Improvement and Deterioration of the Relative Levels after 18 Months on Compacted and Uncompacted Test Sections (Ref. 203)

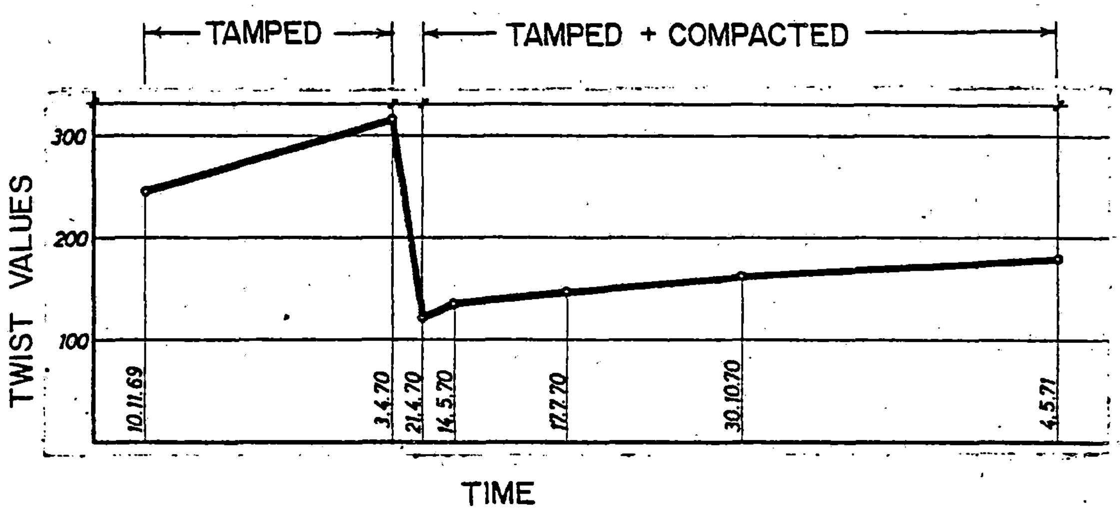


Figure 7.23. Comparison of Track Deterioration After Tamping With and Without Ballast Compaction (Ref. 244)

the time between necessary tamping operations has been quoted for other studies (Ref. 203).

Plasser and Theurer Testing Division (Ref. 247) also reported significantly reduced rate of track deterioration when crib compaction was done on a concrete sleepered track.

CNR (Ref. 137, 192, 255) also noticed that surface improvement seemed to be achieved from compaction. However, CNR (Ref. 137) reported that the degree of improvement in surface obtained from compaction seemed to relate to the initial roughness, and the type of defect causing the original roughness.

Reiner (Ref. 234) also concluded from the track geometry data recorded by a track inspection car in the previously described FRA program, that compaction could reduce the amount of surfacing and lining required, thus lowering maintenance costs.

Other Measurements. There are other types of measurements for evaluating the effects of ballast compaction, such as longitudinal resistance of track, track modulus, dynamic track settlement, and joint profile measurements. Some of these measurements were performed in the previously described FRA program (Refs. 205, 236) in addition to those already described. The results, for example in Fig. 7.24, generally indicated advantages of ballast compaction after tamping.

7.5 SUMMARY

In summary, the results of various research described above indicates that ballast compaction is beneficial, It is equally significant to note that no negative physical results have been reported from ballast compaction. Even though some quantitative results were presented, only qualitative conclusions can be drawn at the present time, because of a large number of parameters influencing the test results. Test conditions varied widely, and most of the

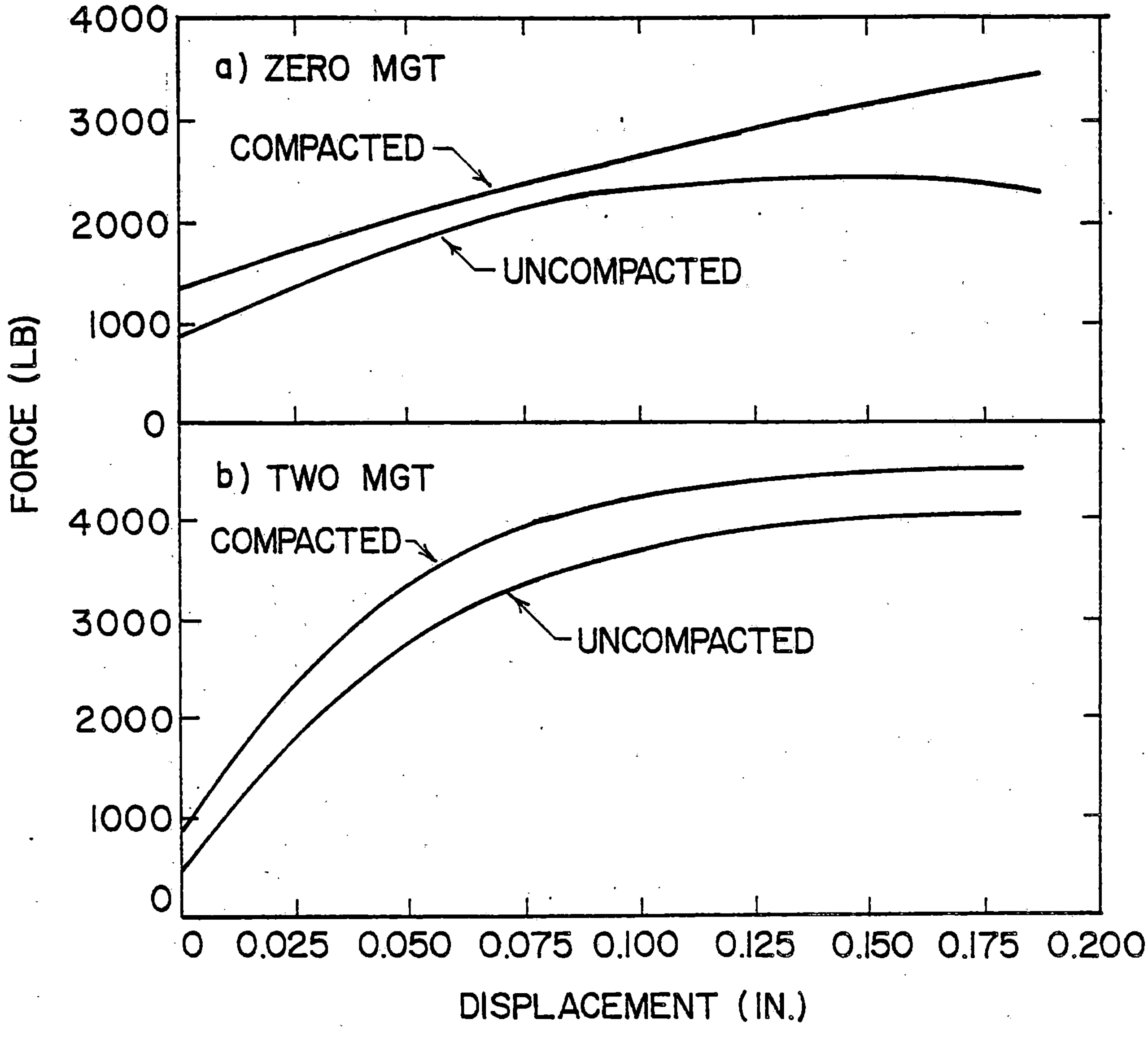


Figure 7.24. Comparison of Typical Longitudinal Tie Resistance between Uncompacted and Compacted Sections (Ref. 205)

reports did not document them adequately. Therefore, a direct comparison of values is not feasible, even between test results obtained under similar conditions. In addition, widely varying test techniques in the absence of any standard procedures makes such comparisons uncertain.

Again, as indicated by Genton (Ref. 201), the problem of ballast compaction must be examined within the framework of an approach that groups those parameters considering 1) the embankment or foundation, 2) an understanding of the track and its constituent elements, and 3) the means and methods of positioning and maintaining the track.

Also, ballast particle size and gradation, the overall dimensions of the ballast section, and existing conditions prior to compaction, such as the quality of tamping and the amount of train traffic from the completion of tamping up to commencement of compaction, should be considered. For example, while a smaller-sized ballast may produce less settlement in service than one composed of large particles, it is equally true that resistance to lateral and longitudinal displacement may be less for the smaller material than for the larger. The configuration of the ballast section, particularly the width and shape of the shoulder, has a substantial influence on the resistance of the track to lateral displacement, even though it has not been fully studied.

It is also well understood that reduced tie spacing could decrease the total and differential settlement of track, for example, as the distribution of traffic load onto the ballast and foundation is distributed uniformly at reduced level.

The quality of tamping plays a decisive role in the efficiency of compacting cribs and even shoulders after tamping, which most of the above studies did not mention. If the tamping equipment is not very efficient, a settlement of the track under the effect of rolling loads may be expected prior to the compaction of cribs and shoulder.

Lining of track prior to compaction may also influence the results of compaction. As Genton (Ref. 201) indicated, improperly lined track may reduce lateral resistance of a track to 30 to 40% for a short period in spite of subsequent compaction.

8. ASSESSMENT OF EFFECTS OF COMPACTOR PARAMETERS

8.1 BASIC PARAMETERS

Ballast compactors presently available in the U.S. simultaneously compact ballast in the cribs on both sides of each tie and/or shoulder area, by exerting static down pressure and a dynamic force. The dynamic force is generated electro-hydraulically with either a rotating eccentric mass or an eccentric shaft drive. Most of the compactors are currently designed to produce a constant static force and a dynamic force with a constant vibration frequency. The values of these parameters often appear to have been more or less arbitrarily chosen. As summarized in Section 6, the ranges vary significantly, the static force from about 1700 to 2200 lb, the rated dynamic force from approximately 1100 to 6900 lb and the vibration frequency from 25 to 75 Hz. However, most of the machines are equipped to vary the duration of vibration.

The main compactor parameters are static down pressure, generated dynamic force, vibration frequency, and duration. These all have important roles in determining the effectiveness of ballast compaction. Other factors are the general characteristics of compactor, the conditions of ballast being compacted, and the foundation supporting the ballast. Ballast compaction under vibrating machines is a complex problem. The ballast and the compactor interact during vibration in such a way that a particular compactor will react differently with different ballast and machine conditions, and hence result in a different degree of compaction.

Ballast compaction with vibration should be examined within a framework that considers all of the important parameters i.e., machine design characteristics, vibration parameters, ballast type and the supporting foundation conditions. However, at the present time there has been no study of ballast compaction that has considered all of these parameters. Important aspects of interaction have been either completely neglected or not understood in past studies of ballast compaction. For example, in an acceptance test of an FRA ballast compactor in which the static and dynamic forces, vertical displacement, and vibration frequency were to be determined, Ensco (Ref. 256) measured the dynamic force while the compacting head or plate was applied to a rubber padded aluminum plate supported on a tie. The value obtained in this manner does not represent the force exerted onto ballast during compaction, and may be misleading as far as the capability of the machine for compacting ballast is concerned, because the transmitted dynamic force is a function of the stiffness of the material being compacted.

Application of vibratory compaction to ballast seems to have originated from the successful use of such methods in compaction of soils, especially granular soils. There have been many studies of vibratory compaction of soils in the laboratory and in the field, even though various misconceptions and uncertainties still exist. Yoo (Ref. 194) has recently presented an extensive summary of the past studies on this subject, and developed a unique theory for vibratory roller compaction of soils which considers the effects of soil-compactor interaction. The study identified the most important factors governing the amount of compaction under a vibratory roller as being the static force, the amplitude of drum motion, and the number of drum oscillations per unit distance of roller travel. Even though the major mechanism of compaction achieved under the ballast compactor may be different from those for vibratory rollers, the basic concept emphasizing the interaction of these parameters could be directly applied to ballast compaction. In fact, as described in Section 5, vibratory compaction behavior of ballast materials has been reported to be the same as other soils having smaller particle sizes (Refs. 160, 161, 190, 191, 193). No differences in the basic phenomena have been observed from various laboratory tests.

8.2 OBSERVED EFFECTS

There is very little field data to assess the effects of various parameters that have been identified as the important ones for ballast compaction. As illustrated previously, the interrelated effects of the various parameters have not been adequately considered for ballast compaction. However, the following discussion is a summary of the effects of compactor parameters on ballast compaction based on the past studies reported so far.

Birmann and Cabos (Ref. 203) investigated the effects on compaction of the compacting head, static down pressure, the rated dynamic force, vibration frequency, duration of vibration, and type of compactor shoes. The compactor tested was a Windhoff crib and shoulder compactor which reportedly generates the dynamic force from an eccentric rotating mass. The tests were conducted in Northern Germany on uniformly tamped straight sections of an in-service line with 23,000 tons of daily traffic. Ballast density was measured at different traffic intervals with a probe-type nuclear gage. No information on ballast type and conditions was reported.

Fig. 8.1 compares density increase from ballast compaction, presumably under the tie, for sections compacted with different static pressures, and for a section tamped but not compacted. Immediately after the tamping, ballast densities of compacted sections show significant increase over uncompacted ones. The difference between lower and higher static pressures is not so evident for traffic of 8 days or less. However, the section compacted with higher

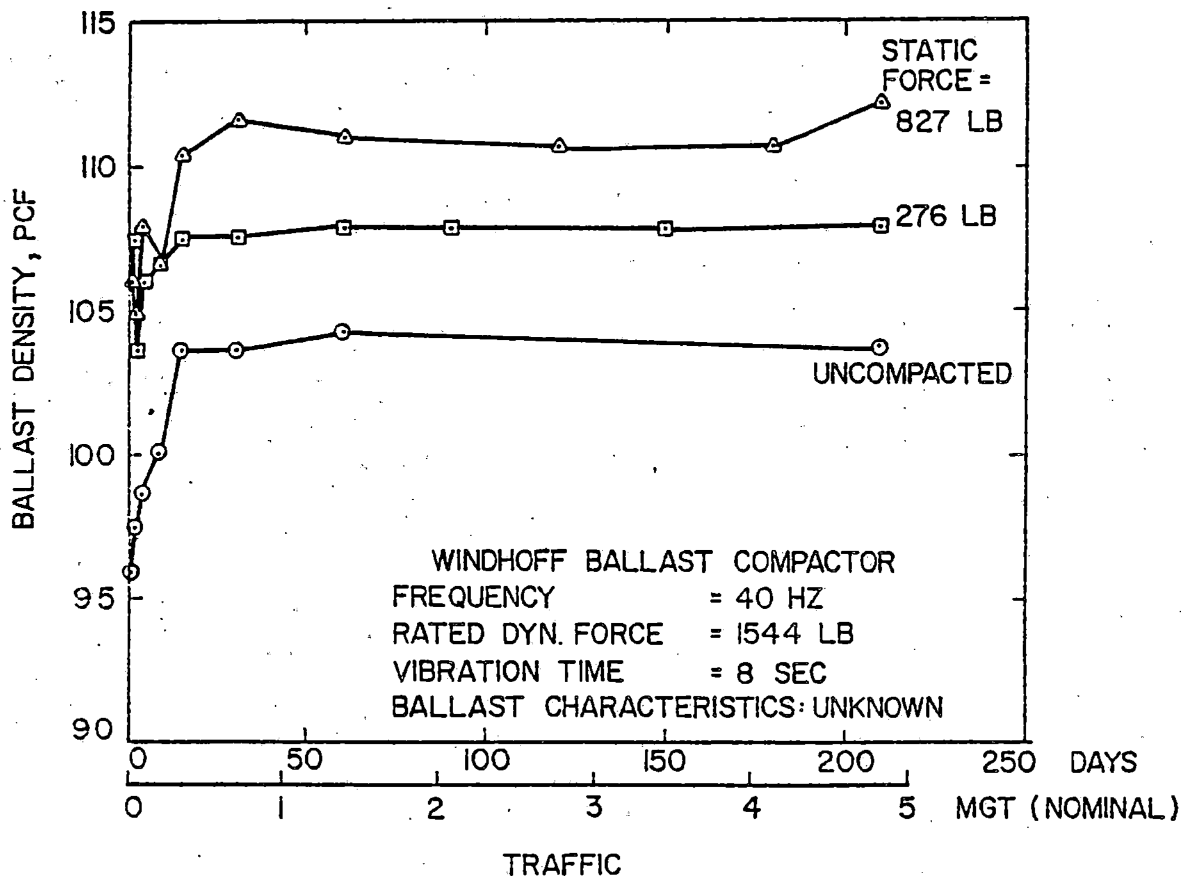


Figure 8.1. Effects of Static Force on Ballast Density (Presumably Under Tie--Replotted Based on the Data in Ref. 203)

static pressure achieved significantly higher densities after 8 days of traffic. The reasons for such a difference upon accumulation of traffic are unknown. However, factors other than the initial compaction conditions apparently influenced the results.

Fig. 8.2 illustrates similar observations made for different magnitudes of dynamic force, apparently the generated force, while other parameters remained the same. Again the effects of the rated dynamic force are not so clear, after a small amount of accumulated traffic, but become evident with further accumulation of traffic. The higher generated dynamic force produced a higher density. It is surprising to note that the amount of density increase with traffic, i.e., the difference between density at zero traffic and the ultimate density achieved under the traffic, was relatively smaller in the case of the lower dynamic force than in the other cases.

Fig. 8.3 shows the effects of different durations of vibration on the ballast compaction. For the tested durations, no apparent difference was noticed in the case of density under the tie. However, it is interesting to note (Fig. 8.4) that in the shoulder area, a shorter vibration time produced a higher ballast density than a longer time.

Birmann and Cabos (Ref. 203) reported that the shape of the vibrator shoes also appeared to have a major influence on compaction. However, no specific data were given to support this claim.

The Plasser Company (Refs. 227 and 251) compared the Plasser VDM 800-U and the Windhoff BV 204, the two major ballast compactors marketed in Europe, to determine the effectiveness of the compactors. The major difference between the machines is probably the mechanism of dynamic force generation. The Plasser machine has an eccentric shaft drive which produces a constant displacement

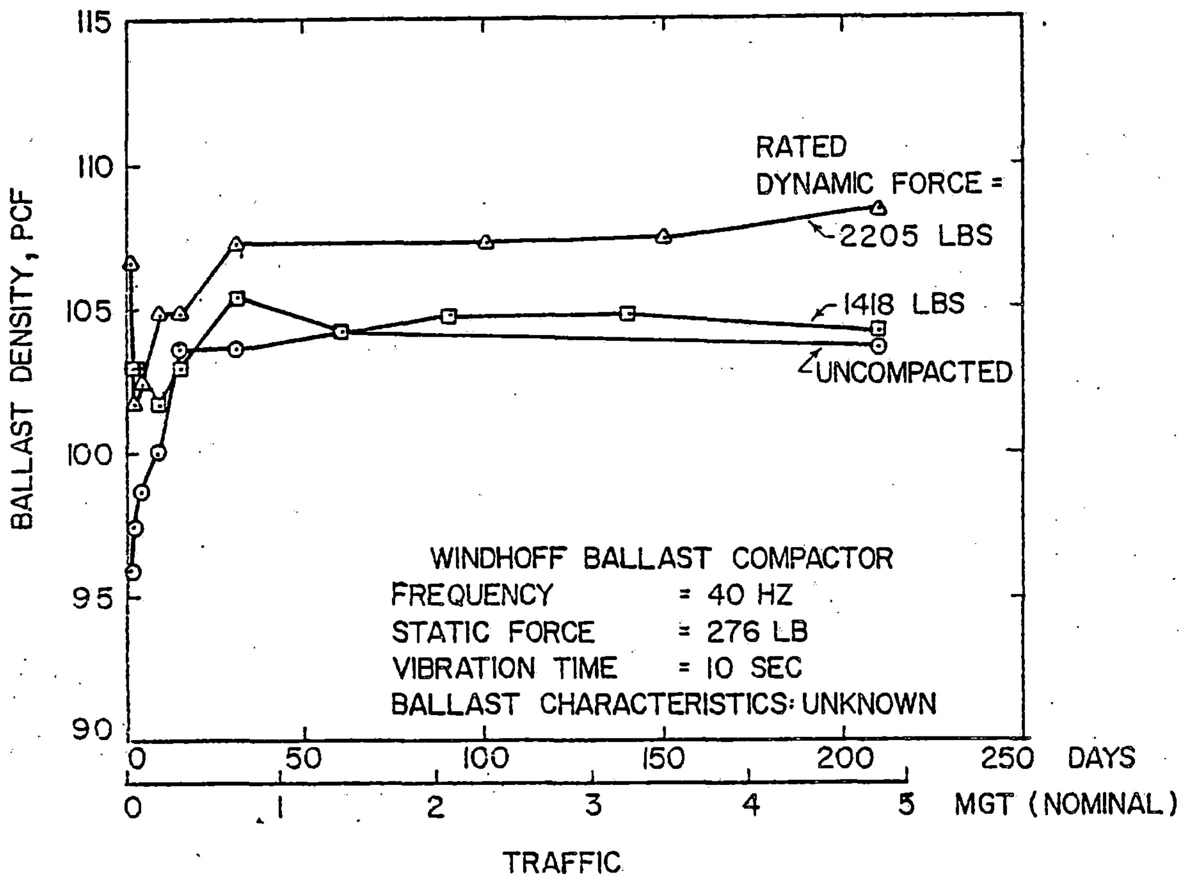


Figure 8.2. Effects of Dynamic Force (Rated) on Ballast Density (Presumably Under Tie--Replotted Based on the Data in Ref . 203)

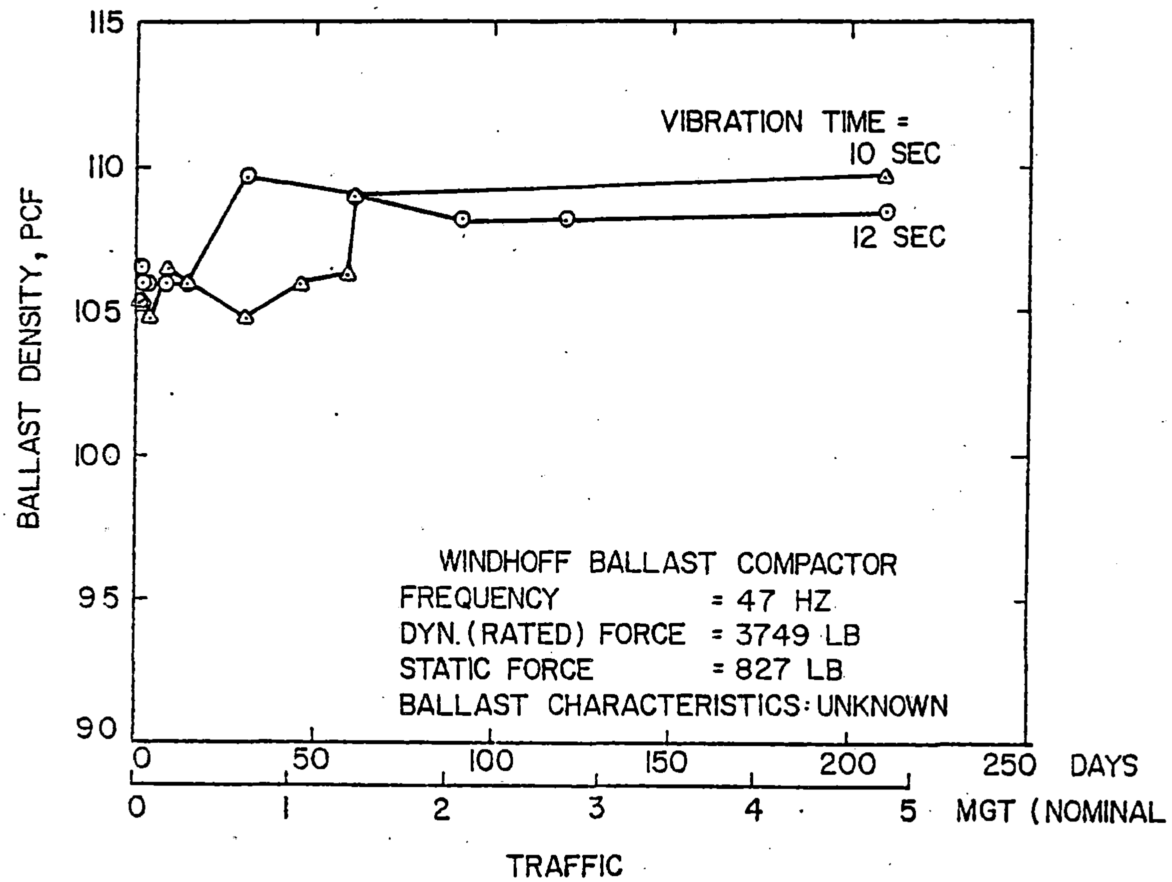


Figure 8.3. Effects of Vibration Time on Ballast Density (Presumably Under Tie--Replotted Based on the Data in Ref. 203)

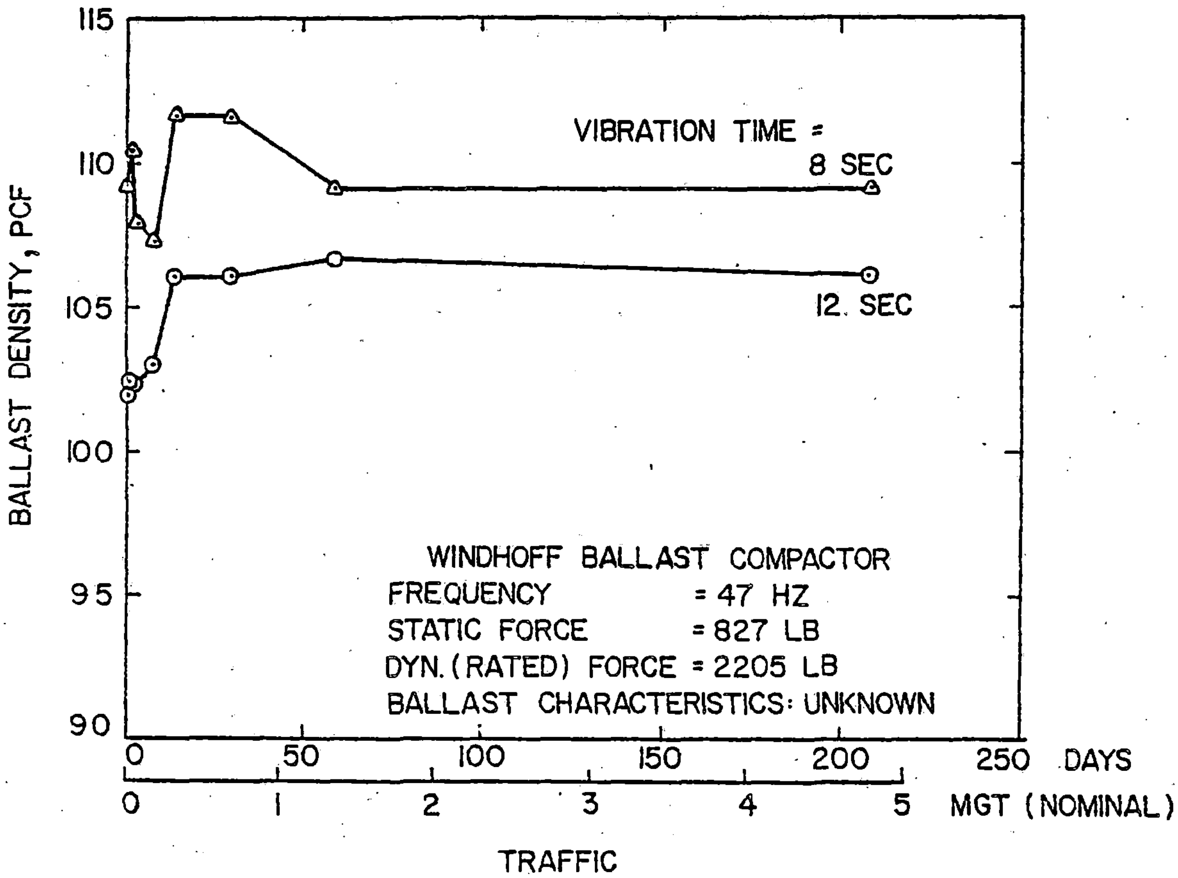


Figure 8.4. Effects of Vibration Time on Ballast Density (Presumably in Shoulder Area--Replotted Based on the Data in Ref. 203)

amplitude during vibration, while the Windhoff machine is based on a rotating eccentric mass, therefore generating a constant dynamic force. The tests were conducted on the Vienna-Graz line at Pottschach, and lateral tie resistance was measured to evaluate the effectiveness of the tested machines.

One of the test parameters studied was the duration of vibration. Each machine was tested using 2, 3, and 4 seconds. The lateral tie resistance as a function of displacement for sections compacted with different durations of vibration are described for the Windhoff machine in Fig. 8.5, for the Plasser machine in Fig. 8.6. Also compared in these figures are the tie resistances for the tamped-only section. As can be seen in the figures, the trends with vibration time are not evident, and different machines show different effects. However, maximum resistance in both cases occurred with 2-second operation. With the Windhoff machine the lowest resistance at 4 mm displacement was observed with 3 seconds of compaction, while for the Plasser machine the lowest resistance occurred with 4 seconds of compaction.

Ensco (Ref. 236) also reported little difference between 3-second and 5-second vibration times from a series of field tests involved with the Plasser machine conducted in 5 railroads in the U.S. The effects of vibration duration, obtained from relevant laboratory vibratory tests (Ref. 193) on samples contained in a mold, in which particle vibration and impact force likely play an important role as in the ballast compaction, are consistent with these observed field results on ballast. The laboratory tests have indicated that density increase during vibration occurs virtually within a few seconds.

Genton (Ref. 201) also conducted field tests with a limited scope at the LIM Access Spur Track at the US DOT Transportation Test Center in Pueblo, Colorado, to find out optimum compaction conditions of a Jackson ballast

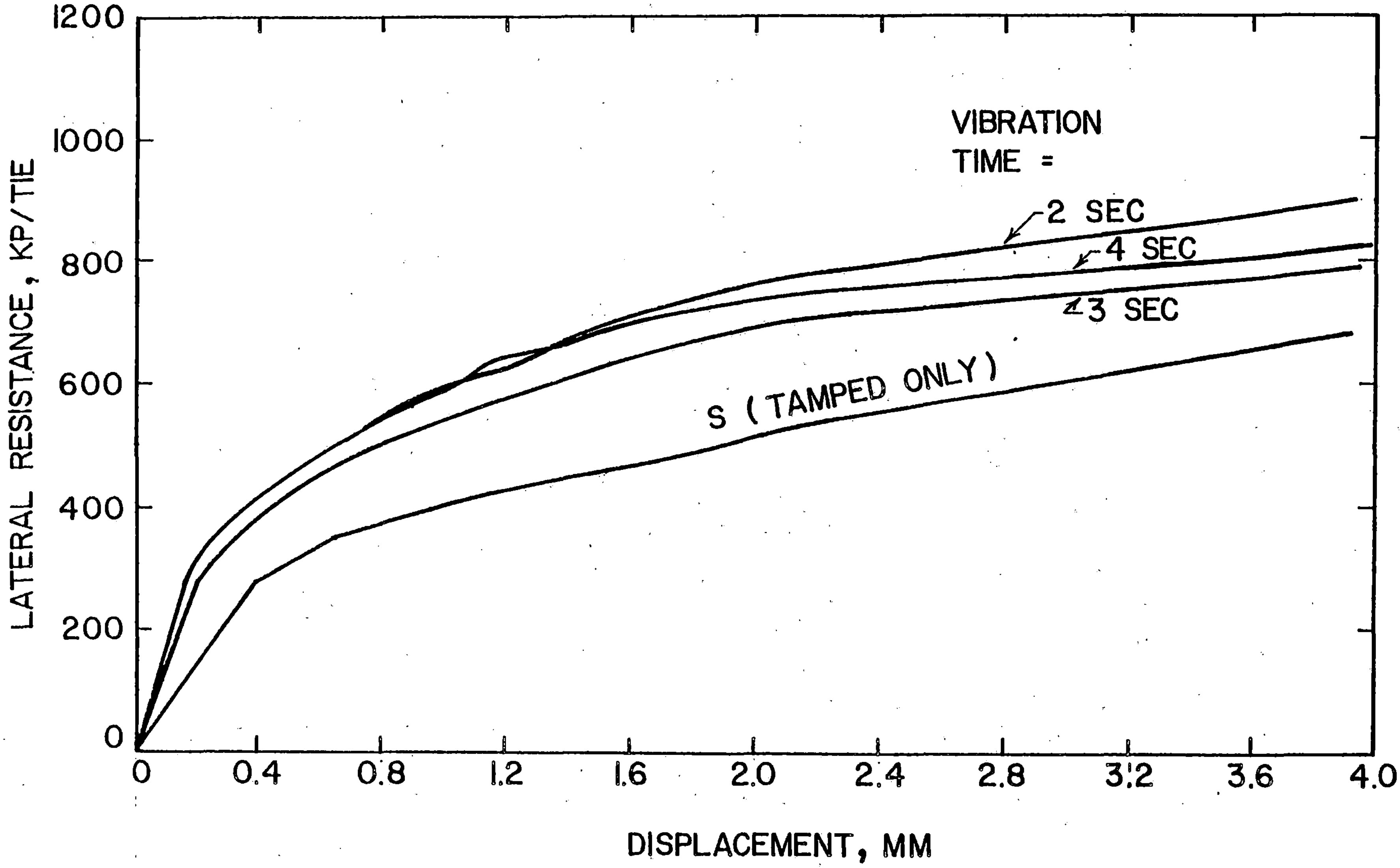


Figure 8.5. Effects of Vibration Time on Ballast Compaction as Determined from Lateral Tie Resistance, Windhoff Ballast Compactor Model No. BV-204 (Ref. 227)

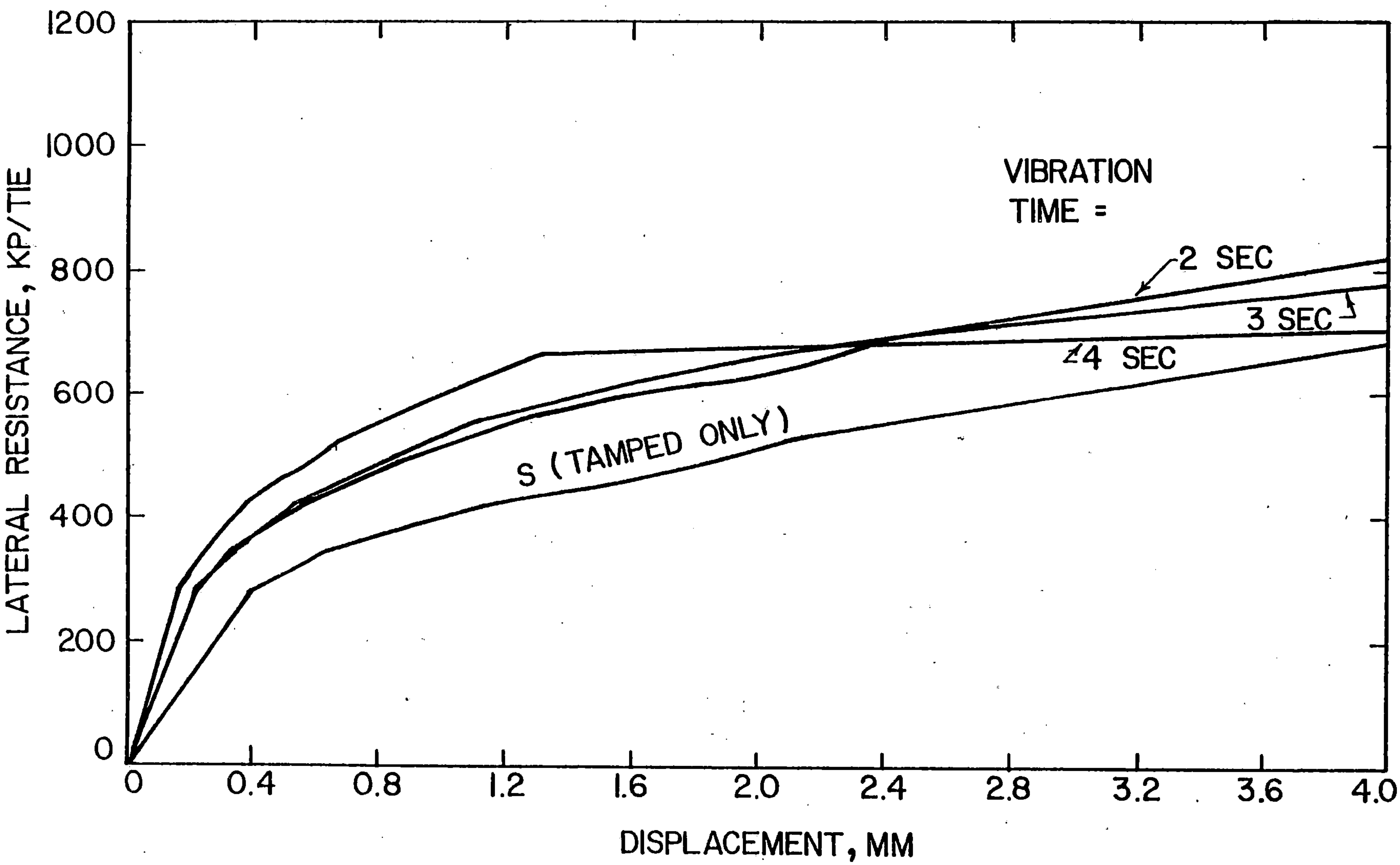


Figure 8.6. Effects of Vibration Time on Ballast Compaction Determined from Lateral Tie Resistance, Plasser Ballast Compactor Model VDM-800 (Ref. 227)

compactor purchased for use in the construction of the LIM Track. Vibration frequency was varied from 50 to 75 Hz, and vibration time from 5 to 10 seconds. The effectiveness of the compactor was determined with rather crude techniques, such as visual examination, comparative sonic evaluation by means of striking the upper tie surface, and depth of compactor shoes imprint. Apparently, Genton observed some degree of ballast particle movement in the crib near the ends of the tie when compacted at high frequency, which he considered as an undesirable condition, therefore discrediting ballast compaction with high frequency such as 70 Hz.

Summarizing the observations made during the above tests, Genton (Ref. 201) suggested that vibration frequency must be in the order of 37 to 50 Hz as a maximum, the duration of vibration should be between 3 and 6 seconds, and the static pressure should range from 14.2 to 21.3 lb/in.^{2}. Genton indicated that excessively high frequencies might cause the lateral flow of ballast particles and destroy the binding or constraining effect of the ballast shoulders. Furthermore, an application period exceeding 6 seconds does not lead to further compaction but, rather, incurs the risk of loosening the tamped regions under the ties. Finally, he indicated that an exceptionally high static pressure may reduce the amount of compaction and increase degradation of ballast particles. However, the validity of these conclusions are questioned because of the test deficiencies as well as contradictions with some other experience in vibratory compaction.

8.3 CONCLUSIONS

The vibration frequency of the ballast compactors currently available in the U.S. and Canada seems to vary between 35 and 50 Hz. There seems to be a general feeling in the railroad industry that higher frequencies than these would cause significant movement of ballast particles and therefore prevent the binding of particles, and that frequencies lower than these may not produce sufficient compactive effort. However, such conclusions are misleading, because frequency is not the only parameter determining the behavior of ballast during compaction.

The static pressure seems to be needed to provide ballast particles with some degree of confinement so that the lateral displacement or free "flow" of the ballast particles being vibrated can be prevented. However, there is plenty of evidence showing that too much surcharge load applied to the ballast particles during vibration may reduce compaction by increasing the interparticle force. It should also be noted that too much pressure onto the ballast particles further increases the possibility of ballast degradation during compaction. Thus, the contribution of the static force to the total amount of compaction should not be neglected. Possibly, the static pressure should be low as compaction begins and then increased as the ballast becomes more stable at the end of an operation at each location.

The rate of ballast density increase is most rapid at the start of vibration and diminishes quickly. Therefore an unnecessarily long duration of compaction, which adversely affects productivity of the compactor, is not warranted. A time of 2 to 4 seconds seems to be reasonable with the currently available ballast compactors. An automatic control could be provided to stop the compaction at a certain ballast resistance, or deformation, for example. However, more research is needed to identify the proper conditions.

The desirable compactor parameters to be used should be determined in each particular case considering the ballast-compactor interaction problems. For this purpose, proper understanding of the compactor behavior during compaction should be gained, and the effects of the influencing factors need to be further studied.

Finally, it should be mentioned that the effectiveness of ballast compaction is also very much dependent on the track conditions prior to compaction, such as quality of tamping, the amount and quality of ballast in the crib and shoulder area, and foundation conditions.

9. ECONOMICS OF TRACK MAINTENANCE

At present, maintenance of way engineers use several schemes by which to plan track maintenance operations. Identification of poor quality track sections either by visual inspection and/or track geometry cars will indicate existing or potential problem areas. This information will either be analyzed by computer methods or evaluated by the maintenance of way engineer to properly decide upon the allocation of the labor force and equipment to the sections of track with a high priority maintenance requirement. The objective is to obtain the most promising cost/benefit ratio or added profit for the added investment by the railroad.

Several factors make economic comparisons of track maintenance practices difficult to quantify. Besides the lack of published cost data, the variations in labor rates, equipment availability and annual budget for track maintenance operations for each railroad add to the difficulty in presenting the economics of track maintenance in the proper perspective. Therefore, the emphasis in this section will be a partial treatment of labor, materials and equipment costs and the associated manpower, quantity and production rates. Both total costs and the individual cost component of labor, material and equipment will be discussed. Presentation will not deal specifically with ballast alone, but will consider the overall track system. Also some of the methods of economic analysis will be presented.

9.1 TOTAL COSTS

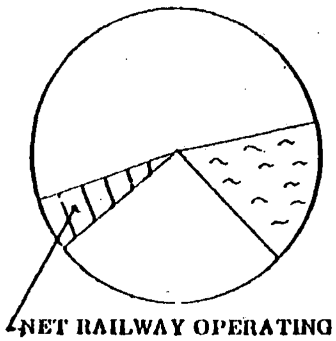
The actual total cost of maintenance is reflected in the difference in cost of conducting traffic before and after changes are made(i.e., track maintenance) on a section of track. Besides the operational benefits of improved track conditions, the benefits from a sales standpoint are also considered. However, only the total cost of the improvement of the track system components will be reviewed here.

Lenow (257) discussed the resurgence of light rail transit in the United States. The author reports that the cost of rail construction varies from $40/ft to $60/ft depending upon the type of material used. The source of this information is not indicated, or whether these values include material, labor, and equipment costs. However, light rail transit vehicles do not exceed 60 ton gross, therefore the track structure is light or, grades steeper, and curves sharper than conventional track. Conventional track is generally considered to be 2 to 3 times more expensive. A general article (258) explains the elements and economics of optimum tangent and curved track maintenance. The article deals with rail maintenance, drainage, stabilization, rail flaw detection, weed control, ballast cleaning, rail installation, rail welding and glued joints.

Punwani, et al. (259) in a report describing recommendations for the proposed Facility for Accelerated Service Testing (FAST) estimates the following benefits from the track tests: FAST will be useful to the railroad industry in improving productivity of equipment and labor through an increase in the life obtained from track structures. By obtaining a more complete understanding of track structure deterioration, the industry will be better able to program maintenance, optimize maintenance cycles and use improved materials that result in lower total life cycle costs.

Cost data are given in Tables 9.1 and 9.2 for U. S. railways. Table 9.1 shows 1973 expenditures for maintenance of way and structures. It is expected that tests conducted in the FAST will have an impact to some degree

Table 9.1. Maintenance of Way Expenditures (Ref. 259)



M of Way
14%
(1,964,292,000)

NET RAILWAY OPERATING INCOME

-7%

(1,037,122,000)

DISTRIBUTION OF RAILWAY OPERATING REVENUES
(1973)

DISTRIBUTION OF MAINTENANCE OF WAY EXPENSES - 1973

ICC A/C NO.			% Total M of Way
	Total M of Way (Frt. Service Only)	1,964,292	100%
201*	Superintendence	188,407	10%
202c	Roadway Maintenance Running Tracks	92,337	5%
212c	Ties-Running Tracks	109,441	6%
214c	Rail-Running Tracks	83,671	4%
216c	Other Materials-Running	89,939	5%
218c	Ballast-Running Tracks	32,929	2%
220c	Track Laying & Surfacing-Running Tracks	424,929	22%
	Total of 201, 202c, 212c, 214c, 216c, 218, & 220c	1,021,553	54%

*No breakdown available between running tracks and other (yard and switches)

Table 9.2. Estimates of Maintenance of Way Costs per Track Mile (Ref. 259)

ESTIMATED	$12,000	RAIL SERVICE IN THE MIDWEST AND NORTHEAST REGION pg 11, Vol I, DOT, 2/1/74: COST ESTIMATE FROM FRA/OFFICE OF ECONOMICS, OR&D, AND OFFICE OF SAFETY
TOTAL MAINTENANCE COSTS PER TRACK MILE =	$ 9,292	MAINTENANCE OF WAY PER EQUATED TRACK MILE (Corrected to 1974 Prices - 5%/Yr.)
(HVY. DENSITY) CLASS 4-5 PER YEAR	$16,720	RAILROAD "A" ESTIMATE ($.50 per 1000 GTM @ 30 MGT DENSITY)

1 mile = 1.609 km
1 Gross Ton Mile (GTM) = 1.46 Metric Ton - km

on 54% of the total amounts expended for maintenance of way. Material costs for track represent 17% of the total maintenance of way and structures budget. Expenditures consist of rail replacement (approximately 6000 miles (9654 km) is projected for 1975), tie replacement (20 million ties projected for 1975), track surfacing (47,000 miles in 1975), and other track materials. Table 9.2 shows estimates of annual maintenance costs per track mile. A 1% increase in productivity of the dollar expended on maintenance of way would result in annual savings of $24 million.

Elements of total life cycle costs for main line track are shown in Tables 9.3. Many potentially beneficial concepts for track structures which have a higher first costs are often rejected or put aside since there is no assurance of increased life or reduced cyclic maintenance costs. FAST will afford the opportunity to ascertain within a relatively short time whether a concept is, in fact, cost effective. It will also afford the opportunity to concurrently test the cost-effectiveness of more than one design or assembly alternative and to demonstrate this in a meaningful way.

Ahlf (260) reports in the work he did on heavy four-axle cars and maintenance-of-way costs, that on the Illinois Central Gulf, 53% of the maintenance-of-way maintenance expense and capital investment is incremental with tonnage; that is, it varies with tonnage hauled. Using the railroad's accounting system, Ahlf concluded the following breakdown of the incremental cost:-

PERCENTAGE	CAUSE	ASSOCIATED COST
36.3%	Rail Deflection	Subgrade stability, ballast, ties, track inspection, lining, and surfacing.
12.5%	Bending Stress	Bolt tightening, turn out renewal, derailment repair, joint renewal, miscellaneous track fittings.

Table 9.3. Elements of Life Cycle Costs for Main Line Track (Ref. 259)

FIRST COST*

I. GRADING & EARTHWORK, DRAINAGE

II. BALLAST - MATERIALS
LABOR
EQUIPMENT

III. A. TIE
B. OTM — MATERIALS, LABOR, EQUIPMENT — PANEL CONSTRUCTION
C. RAIL

D. ASSEMBLY — LABOR — FIELD CONSTRUCTION

IV. TRACK SURFACING & ALIGNMENT

V. BREAK-IN COSTS

VI. DOWN TIME

*Or Present Replacement Value for Present Trackage

INSPECTION FUNCTIONS

VII. TRACK GEOMETRY INSPECTION

VIII. RAIL FLAW DETECTION

IX. A. DRAINAGE INSPECTION
B. BALLAST CONDITION INSPECTION

X. TIE INSPECTION

XI. RAIL SURFACE

XII. A. RAIL JOINT INSPECTION
B. INSULATED JOINT INSPECTION

XIII. DOWN TIME FOR INSPECTION

MAINTENANCE/RENEWAL WORK ELEMENTS

XIV. SPOT TAMPING

XV. SKIN LIFT

XVI. RAIL END WELDING & GRINDING

XVII. JOINT TIGHTENING

XVIII. ODD TIE RENEWAL

XIX. SURFACING & LINING

XX. TIE RENEWAL

XXI. BALLAST CLEANING AND/OR RENEWAL

XXII. ODD RAIL RENEWAL

XXIII. RAIL RENEWAL

XXIV. SURFACING & LINING FOLLOWING RAIL/TIE RENEWAL

DOWN TIME AND BREAK-IN COSTS ASSOCIATED WITH MAINTENANCE AND/OR RENEWAL

XXV. DOWN TIME FOR SPOT TAMPING

XXVI. DOWN TIME FOR RAIL END WELD & GRIND

XXVII. DOWN TIME FOR JOINT TIGHTENING

XXVIII. DOWN TIME FOR ODD TIE RENEWAL

XXIX. DOWN TIME FOR SURFACING & LINING

XXX. DOWN TIME FOR TIE RENEWAL

XXXI. DOWN TIME FOR BALLAST CLEANING AND/OR RENEWAL

XXXII. DOWN TIME FOR ODD RAIL RENEWAL

XXXIII. DOWN TIME FOR RAIL RENEWAL

XXXIV. DOWN TIME FOR SURFACING & LINING FOLLOWING RAIL/TIE REMOVAL

MISCELLANEOUS ELEMENTS OF WORK

XXXV. SLOW ORDER (WORK ONLY)

XXXVI. WEED CONTROL

XXXVII. STABILIZATION OF SOILS

XXXVIII. DRAINAGE MAINTENANCE

MISCELLANEOUS DOWN TIME ELEMENTS

XXXIX. DOWN TIME FOR WEED CONTROL

XL. DOWN TIME FOR SOIL STABILIZATION

MATERIALS COST IN MAINTENANCE

XLI. WELD METAL

XLII. BALLAST

XLIII. TIES

XLIV. TIE PLATES, OTM

XLV. RAIL

XLVI. JOINT BARS, INSULATED JOINTS, ETC.

XLVII. WEED CHEMICALS

XLVIII. STABILIZATION MATERIALS

XLIX. DRAINAGE MATERIALS

11.3%	Contact Pressure	Rail life, rail welding, rail renewal.
40.9%	Supervision & Machinery	Direct relationship to above.

Table 9.4 from Means (261) gives 1977 labor, equipment, and material costs for general site improvement and site clearing. The key items in these categories are tie removal and installation, and surface and lining of existing track. The major cost for the latter is the height of track raise which will dictate the amount of material needed for reballasting. Note that variations in wage rates, labor efficiency, union restrictions and material prices will result in local fluctuations from these average cost figures. Cost indices for construction and labor are available to modify the costs for different geographic regions. It should be noted, however, that railroad labor rates are the same for the whole country and are 15-20% lower than the average construction labor costs.

Hay, et al. (262) reports that the per foot surfacing and lining cost, depending on the combination of machinery and manpower used, is between $0.16 and $0.26 (Table 9.5). This cost is probably somewhat low due to the low machinery costs used.

Table 9.6 from Burns (Ref. 213) gives the associated costs for rail and tie replacements, ballast cleaning, surfacing and clean-up operations. Included are labor force, and type of equipment used. This standard method of track maintenance was compared to the costs of utilizing track renewal trains such as the Matisa P811 or the Plasser and Theurer SUF 500. The costs used are not total costs but relative costs and should be used for comparison purposes only. For example, ballast cleaning cost is $6049/mile but total cost, when items such as signal, switch, and grade crossing are included,

Table 9.4. Total Costs for Site Improvement and Clearing (Ref. 261)*

	Crew	Daily Output	Unit	Bare Costs Mat.	Bare Costs Inst.	Bare Costs Total	TOTAL INCL O & P
HEAVY CONSTRUCTION							
RAILROAD Car bumpers W.D., standard	B-14	2	Ea.	500	270	770	910
W.A., heavy duty		2		850	270	1120	1295
Derails, hand throw (sliding)		10		115	55	170	200
Hand throw with standard timbers		5.5		225	95	320	380
Resurface and re-align existing track		200	L.F.		2.70	2.70	3.60
For crushed stone ballast @ $4.60 per ton, add		500	"	3.10	1.05	4.15	4.85
Siding, yard spur, level grade, 100 lb. rail, new material on wood ties	B-14	59	L.F.	26	9	35	41
Steel ties embedded in concrete		22	"	39	25	64	76
Switch timber, for a #8 switch, creosoted		3.7	M.F.B.M.	365	145	510	595
Complete set of timbers, 3.7 M.F.B.M.		1	Total	1360	535	1895	2215
Ties, concrete 8'-6" long, 30" O.C.		80	Ea.	26	6.70	32.70	38
Wood, creosoted, 6" x 8" x 8'-6", C.L. lots		90		10	6	16	19
L.C.L. lots		90		11	6	17	20
Heavy duty, 7" x 9" x 8'-6", C.L. lots		70		14	7.65	21.65	26
L.C.L. lots		70		15	7.65	22.65	27
Turnouts #8 incl. 100 lb. rails, plates, bars, frog, switch points, timbers and ballast 6" below bottom of tie		23	Ea.	2900	2330	5230	6320
Wheel stops (mat'l. ranges $100 to $400)		14	Pr.	115	38	153	180
EXPLORATION & CLEARING							
Railroad track, ties and track	B-14	110	L.F.	.71	4.14	4.85	6.55
Ballast		500	C.Y.	.15	.90	1.05	1.45
Remova and reinstall ties & track with new bolts & spikes		50	L.F.	1.55	9.15	10.70	14.40
Turnouts with new bolts & spikes		1	Ea.	78	457	535	720
TEST PITS hand digging, light soil	1 Clab	4.5	C.Y.		16	16	22
Heavy soil	"	2.5	"		29	29	40

* O = overhead; P = profit — ≃ 25%; 10% added for subcontractors O & P

- L.F. = linear foot
- C.Y. = cubic yard
- Pr = pair
- M.F.B.M. = 1000' board measure
- Ea. = each
- L.C.L. = less than carload lot
- C.L. = carload lot
- O.C. = on center

Note: Contingency is ≃ 2 to 3%

Table 9.5. Summary of Cost Data for Lining and Surfacing Operations (Ref. 262)

From Generalized Railroad Supplied Data

Machine Organization	Mean Cost ($) Per Productive Hour	Mean Output (Ft) Per Productive Hour	Mean Cost ($) Per Track Foot
Tamper, Liner and one or more ballast regulators	190	720	0.26
Tamper - Liner and one ballast regulator	135	657	0.21
Tamper - Liner, tandem tamper, and one ballast regulator	200	938	0.21

From Detailed Railroad Supplied Data

Machine Organization	Mean Cost ($) Per Productive Hour	Mean Output (Ft) Per Productive Hour	Mean Cost ($) Per Track Foot
Tamper, Liner and one ballast regulator	178	700	0.25
Tamper-Liner and one ballast regulator	122	743	0.16

Table 9.6. The Standard Method (Ref. 213)

750 ties per mile, ballast cleaned, rail changed

Operation	Gang Description or Comments	Labor & Equipment Cost per mile	Man-Days Per mile
Tie inspector	Requires transportation to starting point	$ 27	0.17
Tie train	Ties delivered in weighted average of standard gondolas & special-purpose tie cars (work train crew not included)	1,067	9.18
Tie gang	20-man gang, using tie saw	1,976	20.80
Ballast cleaning	Chain-type undercutter, 700 cubic yards per hour	6,049	25.20
Material distribution	Burro crane, with gondolas, distributing & setting tie plates, anchors, spikes, etc.	2,072	24.00
Rail gang	44-man gang, one rail at a time	7,014	80.52
Final surfacing gang	2 tampers, ballast regulator	785	4.38
Clean up—ties	Only 25% of the ties picked up. Burro crane & air dump cars	333	4.00
Clean up—steel	Burro crane & gondolas	178	1.60
Tie transportation		600	—
Total		$20,101	169.85

Table 9.7. Annual Costs per Mile "Maintenance of Ballast" (A.R.E.A. Report, Ref. 263)

COST OF BALLAST —	MATERIAL	$200
	UNLOADING	24
	CLEANING	104
		$328
LABOR IN DISTRIBUTING		$180
MACHINERY		NOT INCLUDED

is approximately \$12,000/mile, not including depreciation. Since the latter method is yet untried in the U.S., the estimates of the potential savings are uncertain. Details of the cost breakdown may be obtained by consulting the reference.

According to Ref. 212, approximate 1970 costs for Canadian railways are:

1. About \$100,000,000 spent annually, with 40% of this for maintenance of ballast and subballast material.
2. About 100 derailments annually, with a cost of about \$25,000,000 attributed to poor track quality.

Reference 212 also provides a tabulation of the track maintenance cost figures in an appendix based on Ref. 263 prepared from data furnished by the Burlington, C & O/B & O, Chicago and North Western, Illinois Central, Santa Fe, and the Association of American Railroads (AAR). The data were applied to forty-two representative test miles of C & O/B & O track, representing an approximate annual traffic density from one million to 40 million gross tons. Equipment and labor costs are assumed to be included with material costs. The average annual costs for "maintenance of ballast" over the 42 test miles are shown in the attached Table 9.7. Ballast maintenance costs of the CNR are shown in Table 9.8. Neither of these tables contain the cost of ballast transportation which is dependent on distance and can amount to several times the cost of the ballast material. Note, labor and machinery account for about 50% of the total cost.

9.2 MATERIALS COST

Robnett, et al. (1) states that at present, ballast selection criteria are largely based on initial cost, which includes aspects such as availability and transportation costs, but which neglects for the most part, service life and performance level considerations, except as such considerations

Table 9.8. Maintenance of Ballast Costs (Approximate Figures Obtained from CNR, Ref. 212)

	$ PER MILE	$ PER MILE PER YEAR	$ PER YEAR
BALLAST			
MATERIAL-NORMAL RESURFACING	750	187	6.5 MILLION
UNLOADING AND CLEANING*	–	128	4.5 MILLION
TOTAL BALLAST			11.0 MILLION
"OUT-OF-FACE" LIFT			
LABOR	500	89	3.1 MILLION
MACHINERY	–	–	3.1 MILLION
SPOT LIFTS			
LABOR PLUS MACHINERY	–	–	6.2 MILLION
TOTAL LABOR PLUS MACHINERY			12.4 MILLION
BALLAST PLUS LABOR PLUS MACHINERY	–	–	23.4 MILLION

*Obtained separately from A.R.E.A. Report (Ref. 263) 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 structures expenditures. This source of expenditure reflects the frequency with which track smoothing, surfacing, and reballasting operations must be performed. Unfortunately, the relationship between these costs and the degree of stability of various ballasts has not received quantitative analysis.

According to Ref. 1, from an inspection of 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.

Peckover (24) appears to be the only available source which considers the important cost components necessary for ballast selection. The quality of the ballast material, average life of the ballast, average haul distance, material cost and frequency of maintenance cycle were factors included in the analysis. The method is an approximation of real overall costs and can be used in planning maintenance operations. Peckover's (24) conclusion of the analysis is that superior ballast material should be used whenever a long-term cost analysis, i.e., over the anticipated life of the ballast, shows it to be worthwhile. Evidence indicates that long hauls of superior material will frequently be economical. Peckover (24) also emphasizes inventories of future ballast supplies and the development of railway owned sources of ballast.

A 1977 study by Hay, et al. (262) indicates, in the absence of a workable guide, that most railroads base their ballast purchase discussions on price, durability and transportation cost. However, very few railroads quantify even the simple charges for on-line transportation movements and fewer still assign such costs to their maintenance budgets. The report indicates (Table 9.9) that the average purchase price of ballast is between $2.16 and $2.40 per cu yd depending on the type of material. Ballast transportation cost depends on many factors, but the study indicates approximately $0.009 per cu yd per mile for an on-line movement and approximately 5 times this figure for a foreign line movement. The unloading cost was determined to be between $0.47 and $0.84 per cu yd depending on whether unit train or work train delivery is used.

9.3 LABOR COST

Chapel (264) reports on the transition of the railroad industry from manual track labor to mechanized maintenance equipment which usually requires a single operator. The reduction in laborers required, the increased production rates of automatic machinery such as tampers, cleaners, distributors, plows

Table 9.9. Summary of Ballast Purchase Price Data (1975, Ref. 262)

Material Type	Number of Railroads Purchasing this Material	Range in Price ($ per Cubic Yard)	Average Price ($ per Cubic Yard)
Limestone	18	1.02 - 3.35	2.31
Granite	15	1.54 - 3.25	2.36
Blast Furnace and Open Hearth Slags	9	1.65 - 3.00	2.16
All Other Ballasts	16	0.70 - 3.78	2.40

Simple Mean Purchase Price = $2.33 per Cubic Yard

Mean Purchase Price — Weighted by Quantity Purchased = $2.40 per Cubic Yard

Source: Questionnaire

and tie renewal equipment, and the better quality and uniformity of work are all benefits from an economical standpoint. The author also stresses the use of multi-purpose machines to reduce on-track time.

In 1970 Cassidy (212) reported AREA data (263) on labor costs for major lifts and skin lifts (Table 9.10), and Fig. 9.1 shows how labor costs for cyclic track lining and surfacing vary with annual traffic density. Cassidy (212) indicates both the AREA (Table 9.10) and the CN (Table 9.8) agree that the cost of spot resurfacing to correct for differential track settlement after major "out-of-face" resurfacings approaches the cost of "out-of-face" resurfacing. Therefore, improvements in machinery and techniques which result in greater ballast compaction will produce not only better track quality, but reduced maintenance cost as well.

Track laborers and machine operators pay scales for the majority of the railroad industry are set by National Labor Agreement (265). A sample of wage scales for 7/1/76 is shown as Table 9.11.

9.4 EQUIPMENT COSTS

The initial purchasing cost, and the subsequent fuel, oil, machine maintenance and operating costs must be compared to the increase in productivity and economic savings in order to justify the use of a piece of equipment.

The General Managers' Association of Chicago (Ref. 266) provides rates per day for track maintenance machinery rental between railroads. Rates are based on averages from data supplied by member railroads. These average values include depreciation, interest, taxes and insurance, average cost of repairs, and cost of supplies. Operators' wages are not included in the rental rate. Unfortunately few railroads keep accurate track equipment maintenance cost data and coupled with the natural tendency to underestimate costs, the resultant

Table 9.10. Labor Costs for Cyclic Track Lining and Surfacing (A.R.E.A. Report, Ref. 263)

ANNUAL TRAFFICE DENSITY (MGT)	MAJOR LIFT @ $2,090	NO. OF SKIN LIFTS @704	TOTAL COST	CYCLE (YEARS)	ANNUAL COST PER MILE
3	$2,090	3–$2112	$4,202	40	$105
5	2,090	2– 1408	3,498	30	116
10	2,090	2– 1408	3,498	24	145
15	2,090	2– 1408	3,498	20	175
20	2,090	2– 1408	3,498	15	233
25	2,090	3– 2112	4,202	15	281

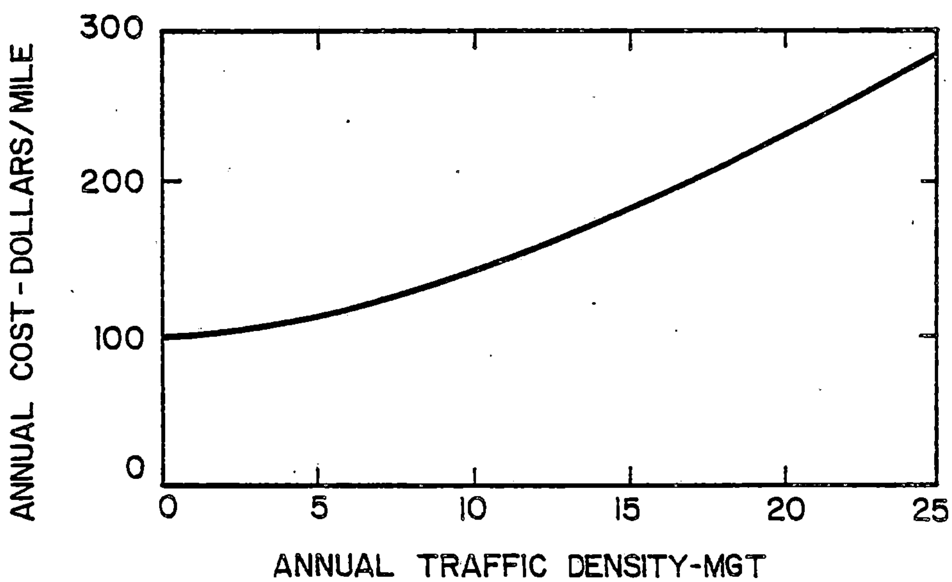


Figure 9.1. Labor Cost for Cyclic Track Lining and Surfacing (Ref. 212)

Table 9.11. Rates of Pay, Including Cost of Living Adjustment, for Track Workers and Machine Operators (Ref. 265)

	7-1-77
Track Inspectors	$7.34
Section Foremen	7.34
Mobile Track Foremen	7.47
B & B Foremen (including Paint Foremen)	7.47
Assistant Foremen (Section & Mobile Gangs)	6.88
Assistant B & B Foremen	7.00
Trackmen	6.22
Group D Machine Operators	6.44
Tie adzer, power rail anchor applicator, bolt tightening machine, cribbing machine, discing machine, Dun-rite gauger, power track jack, rail lifter, track liner, on-track mowing machine, self-propelled ground roller, portable power brush saw, spike driver, spike puller, spike setter, tie creosote spray, chemical or oil spray not self-propelled, tie brush, tie puller and inserter, tractor and tractor mower, wheel tractor with back hoe, tie saw, tie bed scarifier, tie handler, hydra spike, power wrench, rail gauges, trenching machine	
Group C Machine Operators	7.32
Multiple tamper, self-propelled ballast drainage car (head and wing operators), self-propelled CS-6 type chemical spray (head and wing operators), on-track tamping jack, yard cleaner, grouting machine, self-propelled on-track weed burner, track liner Fairmont W-111, chemical spray handled by work train (head and wing operators), boltmaster-jointmaster, ballast compactor, self-propelled brush cutter and ballast regulator	
Group B Machine Operators (Roadway Machines)	7.36
Dragline, angle dozer, bulldozer, borro-crane, front-end loader, clam shell, mobile crane, pile driver, motor grader, ditch operator, Jordan ditcher spreader, shield bantam off-on track crane, multiple purpose loco-crane 66304	
Carpenters	6.95
Carpenters' Helpers	6.46
Painters	6.95
Painters' Helpers	6.46
Masons	7.15
Bridgemen	6.21
Bridge Watchmen	6.04
Whole Line Steel Gang	
Steel Bridge Foremen	8.36
Assistant Foremen	7.96
Steel Bridgemen	6.87
Steel Bridgemen	6.95
Steel Bridgemen (Blacksmiths)	7.32
Steel Bridge Watchmen	6.41
Derrick Engineers	7.47
Derrick Foremen	7.57
Motor Car Repairmen	
Lead Repairmen	7.40
Repairmen	7.32
Repairmen Helpers	6.54
Shop Foremen	7.96
Welders	
Welder Foremen	7.53
Welders	7.53
Grinder Operators	7.18
Welder Helpers	6.54

average figures are considered to be very low. Examination of sample back up data indicates a large variation in depreciation, machine purchase price, maintenance cost, and a general lack in uniformity of railroad supplied data.
A sample listing of rental rates for the various types of track maintenance equipment is shown in Table 9.12.

Railroads, in general, tend to not assign maintenance, fuel, and depreciation to individual machines, but compile the charges under I.C.C. account 269, work equipment maintenance costs. AREA committee 22 developed a method of assigning maintenance cost which, while crude, seems to be more or less valid. The method has yet to be published, but it is as follows:

> The ratio of work equipment maintenance budget (ICC Acct. 269) to undepreciated value of work equipment (ICC Acct. 37) for a particular railroad in 1976 is 0.21 (maintenance budget ratio). It is therefore assumed that the annual running cost of an individual machine is 0.1 times its undepreciated value for a lightly worked machine, 0.2 for an average, and 0.3 for a heavily worked machine.

It has been assumed that all machines operate for 200 single shifts/year.

Therefore

$$\text{Running cost/shift} = \frac{\text{(undepreciated value of machine) (maintenance budget ratio)}}{200} \qquad (9\text{-}1)$$

Per shift fuel and oil requirements have been determined by a similar method and assumed to be: 1) for a lightly loaded machine 0.001 times the undepreciated value per shift, 2) for an average loaded machine 0.003 times the undepreciated value per shift, and 3) for a heavy loaded machine 0.005 times the undepreciated value per shift.

The work equipment maintenance budget covers the mechanics and their trucks that are working with the production gangs. This was the method used by Burns (267) and a sample of the costs calculated are shown as Table 9.13.

Table 9.12. A Sample of Equipment Rental Rates from GMA (Ref. 266)

TYPE OF EQUIPMENT	RATE
Work train on road - 3-man crew	$ 59.75 per hour
4-man crew	68.50 " "
5-man crew	78.00 " "
Ballast cars	7.00 per day
Ballast cribbing machines	
a) Monorail type	16.00 per day
b) Self-propelled, track mounted	26.00 " "
Ballast regulators, conditioners and scarifers	
a) Maintenance Car (W-77 or equal)	63.00 per day
b) Distributor and Cleaner	109.00 " "
Spike drivers	
a) Non self-propelled	14.00 per day
b) Self-propelled single head	15.00 " "
c) Self-propelled single head, automatic feed	83.00 " "
Spike puller	
a) Non self-propelled	8.00 per day
b) Self-propelled	19.00 " "
Tie removers and/or tie inserters	
a) less than 30 H.P.	11.00 per day
b) 30 H.P. and over	43.00 " "
c) production tie injectors	104.00 " "
Tie spacers	43.00 per day
Tie tampers	
a) Hand held, all (including power source)	7.00 per day
b) Spot and/or switch, electronic indicators	102.00 " "
c) Production:	
1) Without jacks or liners	103.00 " "
2) Automatic with jacks only	151.00 " "
3) Automatic with liners and jacks	183.00 " "
4) Switch tampers - high production	224.00 " "
d) Self tamping jacks	63.00 " "
Track brooms; self-propelled	45.00 per day
Track cleaners	
a) Non-track mounted	83.00 per day
b) Track mounted (has car-handling capability)	191.00 " "
Track liners	
a) Without wire indicator, 10 H.P. or less	12.00 per day
b) Complete with wire indicator, over 10 H.P.	33.00 " "
Compactors	
a) Vibratory	4.00 per day
b) Roller type, self-propelled	10.00 " "
Rail layers (not full revolving)	20.00 per day

Table 9.13. Equipment Running Cost Using Maintenance Budget Ratio (Ref. 267)

EQUIPMENT PURCHASE AND RUNNING COST PER 8 HOUR SHIFT

TYPE	TODAYS NEW COST (APPROX.)	COST WHEN LAST PURCHASED	REPAIRS, PARTS LABOR/ SHIFT	FUEL	TOTAL COST PER SHIFT
Air Compressor	$11,000	$ 7,500	$ 5	$ 1	$ 6
Anchor Applicator	12,900	7,900	12	2	14
Automobile	4,500	4,500	12	5	17
Ballast Regulator	55,000	26,900	40	15	55
Back Hoe	29,000	20,500	30	5	35
Bulldozer	67,000	22,500	45	15	60
Burro Crane	148,000	70,000	105	20	125
Carolineum Applicator	3,000		5	1	6
Compactor	80,000	79,000	95	20	115
Gauging Machine	7,000	7,000	12	2	14
Motor Car	4,500	3,000	5	1	6
Motor Grader	57,000	19,500	40	12	52
Rail Lifter	4,300	2,700	5	1	6
Speed Swing Highway Truck Crane	65,000	60,000	90	18	108
Spike Driver	7,600	4,600	8	2	10
Spike Puller (Single)	4,400	2,500	5	1	6
Spiker-Hydro	53,300	31,500	50	10	60
Tamper	60,000	54,000	70	20	90
Tamper-with Jacks	68,000	37,000	150	25	175
Tamper-Production	130,000	121,000	190	30	220
Tamper-Switch	190,000		280	30	310
Tamper-Quadromatic	350,000		515	50	565
Tie Shear or Saw	47,600	27,200	45	8	53
Tie Handlier	22,600	13,400	20	4	24
Tie Bed Scarifer	32,000	20,700	35	6	41
Tie Inserter	8,300	4,200	7	1	8
Tie Cribber	7,200	3,400	6	1	7
Tie Adzer	14,800	7,300	15	2	17
Tie Plug Machine	8,100	3,900	6	1	7
Truck	12,000	12,000	40	10	50
Truck Crane with Clamshell	77,000	34,500	55	10	65

It is interesting to note that for a production tamper, GMA cost including depreciation is $183, the AREA method which excludes depreciation is $220, and a construction contractor would charge approximately $500 per day for a similar machine. A further means of equipment comparisons would be based upon the rated productions listed in Section 6.

9.5 METHODS OF ANALYSIS

The technique used to compare the effect of the alternative sections of track selected for maintenance on the company's total operation is the discounted cash flow analysis. Here the project costs are weighed against financial benefits over the life of the project. Since numerous equations are used in this type of analysis, consult Grant et al. (Ref. 268) for further details. This method is also suggested by AREA (Ref. 2).

An example of this method is given by Dunn, et al. (269) which was used to evaluate different rail sections, tie types and tie spacings. The portion of this analysis dealing with track maintenance was taken from AAR (Ref. 270, 271).

The following formula developed by Magee (Ref. 271) may be used to approximate annual track maintenance costs per mile of track constructed of 90 lb rail with train operating speeds of 50 mph. It is based on current track maintenance experience as reported by American railroads.

$$M_c = 982\ (B)^{0.282} \quad , \qquad (9\text{-}2)$$

where: M_c = Annual track maintenance cost per mile of single track, and

B = Annual traffic density (MGT).

The value of M_c varies inversely as the square root of the moment of inertia of rail, i.e.

$$M_c \approx \frac{1}{I^{0.5}} \quad , \qquad (9\text{-}3)$$

where I = moment of inertia of rail (in.^{4}).

To compute M_c for any other rail section with moment of inertia I, multiply the right-hand side of the square root of the moment of inertia of 90 lb rail to the square root of the moment of inertia of the rail section desired, i.e.

$$\frac{(I_{90})^{0.5}}{I^{0.5}} = \frac{6.22}{I^{0.5}} , \qquad (9\text{-}4)$$

where I = Moment of inertia of rail section desired.

To compute M_c for any other operating speed, multiply the right-hand side of Eq. 9-2 by the ratio of the speed desired (mph) to a speed of 50 mph.

Applying the appropriate factors to Equation 9-2, gives

$$M_c = 982(B)^{0.282} \left[\frac{6.22}{I^{0.5}}\right] \left[\frac{V}{50}\right] , \qquad (9\text{-}5)$$

Reiner (272) presents estimates and approximations with regard to the magnitude of the penalty that maintenance-of-way departments must pay in the area of routine maintenance as a result of operating jumbo cars, i.e. 90 to 125 ton capacity cars. The average freight car capacity is 65 tons. The increase in total load also brings about an increase in wheel loads and wheel-rail pressures. Besides the desire to compete better with other modes of transportation, the higher capacity cars yield a more favorable payload/tare weight ratio. However, increasing wheel loads cause maintenance problems, the most serious of which was said to be the rapidly growing rates of deterioration of the rails.

According to Ref. 272 the additional maintenance-of-way and structure costs associated with heavy wheel loads can be classed in two groups: 1) additional investments, such as costs of strengthening roadway and increasing clearances, and 2) increased cost of routine track maintenance which mostly consist of cost elements which vary with the traffic volume. Methods of estimating incremental cost factors are described in Ref. 272.

Reiner (273) utilizes a systems approach in the determination of the proper wheel load to yield maximum profit. This is qualitatively shown in Fig. 9.2 which includes the pertinent parameters affecting profit. The author uses the relative cost factors for rail, crossties, and surfacing and lining described in Ref. 272).

Danzig, et al. (274) builds on the work done by Reiner and attempted to determine specific costs of various track maintenance functions as opposed to the averaging method generally used. This was accomplished by applying predictive life cycle relationships to specific track/traffic conditions. The study includes the predictions of various track component service lives, including that of a surfacing cycle. The study assumes no deferred maintenance and a good tie condition.

Rail life was determined to conform to the following formula:

$$T = \frac{K \quad W \quad D^{1.565}}{\sum_{i=1}^{m} \sum_{i=1}^{n} \sum_{k=1}^{r} \left[\frac{D_{ijk}}{F_{ijk}} \right]} , \qquad (9\text{-}6)$$

where T = total main line track rail life in million gross tons,

D = total annual tonnage in million gross tons,

W = weight of rail in pounds per year,

K = relative life factor which reflects the physical condition associated with the track structure (This factor is the product of applicable curvature, gradient, rail metallurgy, and a factor related to whether the rail is continuously welded or jointed. These various factors are multiplied together to arrive at the overall relative life factor),

F_{ijk} = multiplier which relates the effect upon rail life of type i wheel loads carried at class j speeds, in train service type k (This factor is the product of applicable wheel loads, train speed, and train service), and

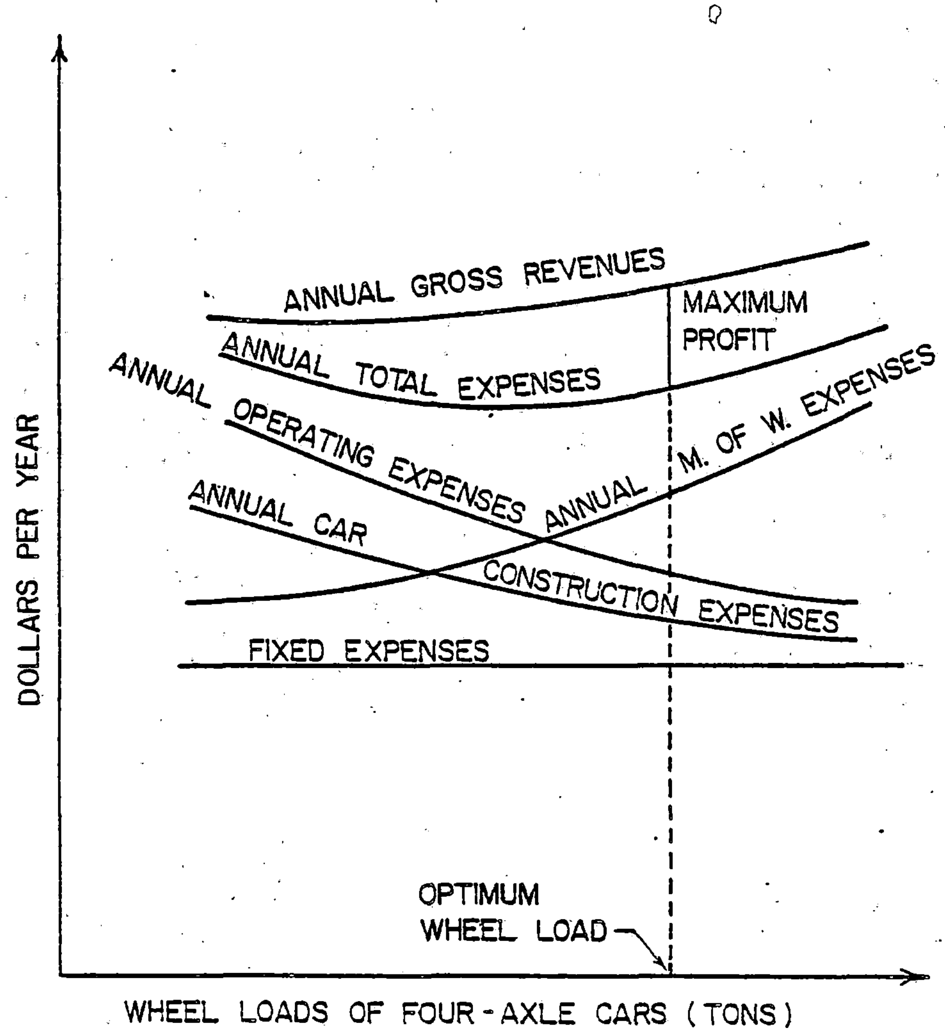


Figure 9.2. Typical Relationship for Costs, Revenues, and Profit vs. Wheel Loads (Ref. 273)

D_{ijk} = annual tonnage carried in type i wheel loads at class j speeds, in train service type k.

The relative life, speed, wheel load, and train service factors were determined largely by careful observation. For table of the various factors, consult Ref. 274.

Tie life was determined to confirm to the following formula:

$$H = \frac{L \quad D^{0.816872} \quad e^{(4.0925-0.06077d)}}{\sum_{i=1}^{m} \sum_{j=1}^{n} \sum_{k=1}^{r} \left[\frac{D_{ijk}}{F_{ijk}}\right]}, \tag{9-7}$$

where H = tie life in years,

e = base of natural logarithms,

D = total annual density in million gross tons,

L = relative life factor which reflects the influence which tie plate size, rail type (CWR, jointed), gradient and track support quality have upon tie life,

F_{ijk} = multiplier which relates the effect upon rail life of type i wheel loads carried at class j speeds in train service type k (This factor is the product of the applicable wheel loads, train speed, and train service), and

D_{ijk} = annual tonnage carried in type i wheel loads, at class j speeds, in train service type k.

The various multiplier factors were determined by observation and detailed analysis of 10 tie renewal projects.

The following basic surfacing cycle equation (Eq. 9-8) was developed to reflect the same track/traffic conditions attendant to the basic equations developed for rail and ties. The frequency for surfacing was found to conform to the following:

$$J = \frac{M \quad D \quad 10^{\left(\frac{121.31-D}{149.5}\right)}}{\sum_{i=1}^{m} \sum_{j=1}^{n} \sum_{k=1}^{r} \left[\frac{D_{ijk}}{F_{ijk}}\right]}, \tag{9-8}$$

where J = surfacing cycle length in years,

M = relative cycle length factor which relates the effects of gradient, track support quality, and rail weight on surfacing cycle length,

D = total annual density in million gross tons,

F_{ijk} = multiplier which relates the effect upon surfacing cycle, length of type i wheel loads, carried at class j speeds in trains service type k, and

D_{ijk} = annual tonnage carried in type i wheel loads, at class j speeds, in train service type k.

The relative multiplier factors are the same as used for the rail and tie life with the exception of train service type. Passenger trains and unit trains reduce the cycle lengths by 20% and 10%, respectively.

The study also developed unit cost formulas for undercutting, plowing, and sledding, and for turnout, and track crossing replacement. The cost of a maintenance operation is relatively simple to determine, but must be calculated with reference to such things as geographic location, methods, and equipment available. The study attempts to determine accurate component life or cycles. It is, therefore, a relatively simple matter to then determine the various aspects of track costing in terms of tonnage, speed, and axle loading.

10. SUMMARY

The subject of ballast compaction and its influence on track performance encompasses a variety of specific, related topics. Each of these topics has been individually treated in sufficient detail to present their relative importance and contributions to the ballast compaction-track performance problem. A comprehensive summary and the associated critical assessment of the reviewed material will be presented in this final chapter.

The performance of conventional railroad track systems is directly a function of the complex interactions and subsequent responses of the track system components under train traffic and environmentally-induced stresses. Descriptions of the track structure have been presented, including the rails, ties, and fasteners, and in particular, the ballast, subballast and subgrade. Also, the relationships which exist between the track system components and track performance have been demonstrated. The ballast and subgrade behavior with the associated in-situ physical states, especially those existing after programmed track maintenance operations, is the most significant contributing factor which affects the performance of the track in-service. Recently developed analytical track models have been described, with particular emphasis concentrated on the representation of the ballast and subgrade materials. However, the reliability of these techniques for predicting track performance was not established. The primary reason is the lack of corroboration with field data. This situation, however, will be improved when the generated substructure stress and strain measurements from the field instrumentation installed at the FAST track are thoroughly analyzed and compared to the predicted responses. Other available field measurements or criteria have been described for use as indicators of track performance. These are track stiffness, track geometry, safety, ride quality, and maintenance effort. However, each individual item is not sufficient for proper representation of the overall track system performance.

Ballast materials, as derived from various rock sources or from by-products of manufacturing processes, also inherently possess different particle physical and chemical properties. Many laboratory index property tests, such as those for abrasion resistance, absorption, shape and soundness, are currently utilized to quantify and categorize the relative merits of these different ballast types. The applicable test standards and ballast specifications limits have been cited and the basic test procedures, as well as the factors influencing the test results, have been discussed. However, an individual index property test by itself cannot clearly be used as a direct indicator of expected field performance of the ballast matrix in the track structure under traffic loading and environmental conditions. This latter feature is particularly evident from the differences in published opinions, which in fact are primarily subjective interpretations, as to what values of what index properties represent the best ballast.

Granular materials, which include ballast, are typically categorized as discrete particulate systems having discernible particle dimensions, ranging from sand to boulder sizes. The fundamental soil mechanics principles representing strength determination apply equally well to all granular soils. However, most of the available information defining the characteristic static or dynamic stress-deformation behavior is related to sand-size - particles and is determined from laboratory property tests. Alternative methods based upon shear strength parameters obtained from modelled gradation curves have been proposed as a viable means of estimating the properties of larger-sized materials. However, these techniques have only a limited amount of supportive data. Therefore, the behavior of ballast-sized materials must be determined directly from laboratory strength tests, in lieu of applying previously published strength information on granular materials. The conventional static and dynamic test apparatus and procedures, as well as the factors affecting the test results, have been discussed and assessed with respect to use with ballast materials. The cyclic or repeated load triaxial test offers the greatest potential for realistically determining the ballast behavior, since the imposed stress conditions and type of drainage can be controlled for specimens prepared at given levels of compaction. Ballast behavior, as determined from laboratory tests, needs to be supplemented with in-situ strength data on the ballast physical state. An evaluation of the available field methods indicates that the plate bearing test is best suited for this purpose.

The strength and compressibility characteristics associated with the relative degree of compaction of ballast within the track structure are directly related to the levels of track performance, in at least a qualitative sense. In the past, for most geotechnical engineering applications, this relative degree of compaction, as used as a means of field compaction control, has been most easily and conveniently determined by comparing in-situ soil moisture-density measurements with the appropriate laboratory compaction test results. Several laboratory compaction methods have been developed in an attempt to simulate the characteristics or the anticipated end-product results of conventional field compaction equipment. The test specifications, test procedures, and factors influencing the test results for these laboratory methods, such as impact, kneading, static, and vibratory tests, have been discussed for granular materials. However, these methods have limited application for determining the compactibility of ballast materials. At present, no quantitative ballast compaction specifications are available. Furthermore, measurement of the degree of ballast compaction is rarely done. However, an in-situ ballast density test, a small plate load test, and a single lateral tie push test on an unloaded tie have been identified as potential means of representing the ballast physical state.

The rate of change in the ballast physical state within the track structure caused by train traffic loading and environmental conditions is highly dependent upon the initial state achieved from normal track maintenance operations. Ideally, an "undisturbed" ballast trackbed after considerable traffic is the most desirable condition from a stability viewpoint. The reason is that the subsequent correction of track geometry defects with mechanized track equipment inevitably disturbs the stable condition created by traffic. Therefore, the effect that present track maintenance processes have upon changing the ballast physical state requires an equal amount of consideration. To aid in understanding these effects, the principal types of track maintenance equipment and the associated field procedures in current use have been described.

Several field methods have been devised as measures of track performance, but more specifically each has been used as an indicator of the benefits of adding ballast compaction after the track tamping, leveling, and lining operation. The test equipment, test procedures, and typical observed trends obtained with these field methods have been reviewed and presented, with the emphasis placed upon ballast compaction. The ballast state produced by the tamping, leveling, and lining operation and the more recent ballast crib and shoulder compaction process after tamping, are discussed in relation to their expected effect on track performance under traffic following maintenance.

Lateral resistance of single tie and multi-tie track sections is the method most frequently used for measuring track performance, but alternative means including track geometry, track settlement, vertical ballast stiffness, and ballast density measurements appear to be gaining popularity. A broad and detailed interpretation of all results is limited because several factors, such as the differences in testing equipment and techniques, and test conditions, exercise some influence on the measured results, and each method in itself is not indicative of the overall track system performance. However, in a qualitative sense, the supporting evidence and published opinion appear to indicate benefits of mechanical ballast compaction following the usual tamping operation.

The amount that the ballast is compacted with crib and shoulder compactors is a complex function of ballast-compactor interaction. Besides the initial ballast physical state and other in-situ track conditions, the compactor characteristics of static force, generated dynamic force, vibration frequency, and duration of vibration have an influence on the results. However, insufficient information is available to predict their quantitative effect on ballast compaction. It is therefore imperative that more thorough and controlled laboratory and field investigations identify the relative importance of the factors influencing ballast compaction to find the most effective means of obtaining the desired physical state of ballast.

The relationships between the ballast compaction parameters and the track performance level parameters are quite complex, and hence an economic assessment is difficult. Limited published information was available on costs related to track maintenance, such as materials, labor, and equipment, and significantly less with regard to ballast crib and shoulder compaction equipment. In addition, the several methods of economic analysis discussed for track maintenance, although superficially treated, are dependent upon the geographic location and the in-situ track conditions. Thus, they are of limited capability in assessing the actual cost-benefit relationships derived from ballast compaction. The economics associated with ballast compaction, as with other major track maintenance operations, are as difficult to evaluate as the mechanics of ballast compaction and, thus, deserve an equally important treatment.

The practices and principles of geotechnical engineering provide direct and valuable input for understanding the behavior of the ballast, subballast and subgrade within the track structure from the imposed loading environment. But due to the variability of the train loading conditions and the associated environmental influences, as well as the changes in ballast physical state with track maintenance operations such as out-of-face surfacing and lining and crib and shoulder compaction, the development of a quantitative understanding of the mechanisms causing ballast compaction is inherently more complex. The lack of in-situ measurements, and the limited applicability of laboratory property test results and computer-oriented track design models, still prohibit a thorough understanding of all the factors influencing these ballast compaction mechanisms. Thus, further research is necessary in order to reasonably establish a foundation toward these goals.

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APPENDIX A

REPORT OF NEW TECHNOLOGY

The work performed under this contract does provide a needed summary of ballast compaction and related topics existing prior to 1979. Available information on the effects of parameters influencing densification of ballast with vibratory crib and shoulder compactors was shown to be inadequate to determine the most effective combinations. A lack of methods for rating ballast for its influence on track performance was noted. The need for methods to measure the physical state of ballast in track was identified.

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